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Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

FREEZING OF SLURRY AROUND PILES IN PERMAFROST

Ronald F. Scott[1]

SYNOPSIS

In Arctic construction, piles are frequently placed in drilled holes in permafrost and held in position while a slurry of sand or silt and water is poured into the annular space around them. This slurry, under the influence of below-freezing temperatures in the surrounding ground, eventually freezes. The paper presents an analysis of the problem involved in predicting the time that a slurry will take to freeze in particular conditions. The problem was solved by means of an electronic analog computer, and curves of the solution are presented.

INTRODUCTION

The work described below was carried out on the electronic analog computer of the Arctic Construction and Frost Effects Laboratory, New England Division, Corps of Engineers, U. S. Army, as part of a program of investigations into heat flow phenomena involving freezing and thawing in soil. Since such problems are very complicated mathematically, and only a few exact solutions have been obtained in simple cases, the use of approximate methods of solution, such as the computer, is mandatory for practical problems.

Description of Problem

Because of the difficulty of driving piles into perennially frozen ground in Arctic and subarctic areas, a different technique of placing them is commonly

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used. In this method, a hole is excavated in the ground, the diameter of the hole being only a few inches larger than that of the piles; then the pile is inserted in the hole, and an unfrozen slurry of sand and water is placed around the pile. When this slurry freezes, the pile will be firmly held in place, and construction of the supported building can commence. Details of this construction method appear in various reports of the Arctic Construction and Frost Effects Laboratory.(1)

If the undisturbed temperature of the surrounding soil of the pile is sufficiently far below freezing, the slurry will freeze naturally in a few hours. However, if the soil temperature is close to (but below) freezing, it may be necessary to install artificial refrigeration around the pile in order that freezing of the slurry can take place in a reasonable time. In this instance, of course, the refrigeration piping must be left in place, and the cost of the pile installation is considerably increased.

As it is desirable to be able to predict approximately the time that a given slurry in a given ambient ground temperature will take to freeze, it was decided to carry out a series of solutions to the problem on the electronic analog computer at the Arctic Construction and Frost Effects Laboratory. This report describes the assumptions and approximations made in order to obtain the solutions, and presents the results of the investigation.

Simplifications and Assumptions

In the undisturbed ground, temperatures vary with depth and with time, depending on the surface conditions and on the thermal properties of the soil. In the summer, when construction usually takes place, soil temperatures in permafrost will increase from near the annual mean (below freezing) at depths of from 20 to 30 feet, to temperatures generally above freezing at the surface. When the hole for the pile is bored, the natural thermal regime in the ground is disturbed and some warming of the soil around the hole is bound to take place. The warming is largely due to the introduction of air by the drilling equipment, but this effect is probably small, since the time between drilling the hole and placing the pile and slurry is usually short.

Sometimes the drilling of the hole is facilitated by the use of steam to thaw the frozen ground, and, in such circumstances, the soil temperatures will be considerably modified before the slurry is placed. This procedure is not usually recommended in locations where the mean annual temperature is close to 32°F., unless artificial refrigeration is to be used to freeze the slurry, since, without this aid, the slurry may take several months to freeze. No records of the use of steam-thawing in areas of low mean annual temperatures are available. In many instances, most of the temperature disturbance is due to the sudden placing of the slurry, at a temperature of a few degrees above freezing, in contact with the soil around the drill-hole.

As the slurry is all at the same temperature initially, and the soil temperature varies with depth, the problem to be solved is a three-dimensional one, involving both radial and vertical heat flow. However, it is likely that most of the heat flow will take place in the radial direction and not vertically, since the slurry temperature will generally drop very quickly to the freezing point at all depths in the frozen soil, and thus little vertical heat flow will occur in it. Although vertical heat transfer will have some effect on the temperature of the surrounding soil, it is felt that the time which an annulus

of slurry at a particular depth will take to freeze is influenced predominantly by the temperature of the ground at that depth.

The problem was therefore resolved into one involving one-dimensional radial flow in a sector of a circle containing a pile at the center, the slurry next to it, and the natural ground beyond the slurry, as shown in Fig. 1. Then, in effect, the natural homogeneous ground in this simplification is assumed to be initially all at one temperature below freezing, and its inner boundary is suddenly placed in contact with the slurry which is slightly above freezing. The final temperature of the pile slurry and natural ground will be the same and equal to the initial ground temperature.

The thermal conductivities and volumetric heats of slurry over a range of water contents are not far different from those of various natural soils, and it was found by trial that the solution was not significantly affected by assuming that the slurry conductivity and volumetric heat were the same as the values in the surrounding soil. In preliminary tests of the problem, it was discovered that the effect of the thermal properties of a wood or concrete pile on the solution was negligible, and, accordingly, the pile was considered to be a perfect insulator. It was also found that the effect of the presence of the slurry did not extend, within the range of measurement of the apparatus, beyond a distance of about five times the radius of the hole from the center of the pile during the time that the slurry was freezing for all the ambient ground temperatures examined.

The method adopted in the simulation of the problem is the now well-known use of "lumped" parameters. The field of the problem is divided up into a convenient number of lumps, and the thermal properties of resistance and capacitance and latent heat for each lump are simulated on the computer. It has been demonstrated previously[2] that the simulation of a sufficient number of lumps can yield results very close to the correct or analytical answer to the problem, when one exists. The lumped problem is shown in Fig. 1.

Several other assumptions are implicit in this method of solving the problem. It is not possible, without complicating the situation to an undesirable degree, to take into account the gradual development of ice in the slurry, in layers next to the wall of the hole. It is assumed, therefore, that the slurry cools to 32°F. in a negligible time, and that heat is abstracted thereafter from the slurry mass until, at an instant, the slurry freezes homogeneously when all its latent heat of freezing has been removed. No thawing in the natural soil next to the slurry is presumed to take place. Soil moisture is assumed to freeze at 32°F.

On the basis of these approximations, the problem was set up and solved on the computer.

Analog Solutions

The solutions are presented in the accompanying Figures 2 and 3, which give the approximate time that a slurry will take to freeze in a given ambient temperature. An explanation of the derivation of the dimensionless parameters is given in an appendix.

In order to use Figure 2, it is necessary to know the density and water content of the material at different depths in the soil profile at the site, and the thermal properties can then be estimated. With a knowledge of the temperature profile prevailing in the ground before the drilling of the hole, an ambient temperature at any one depth can be found, which, together with the

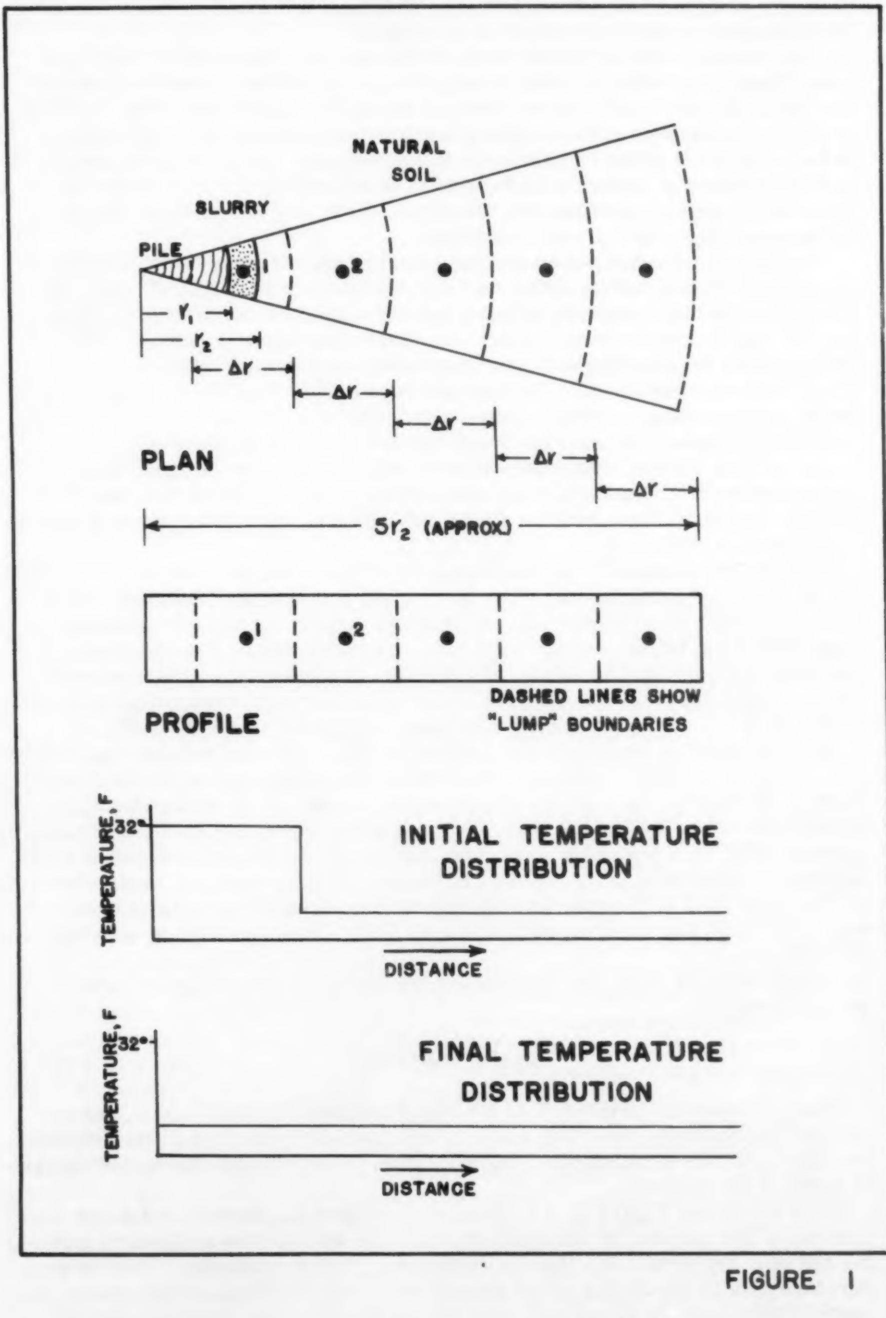
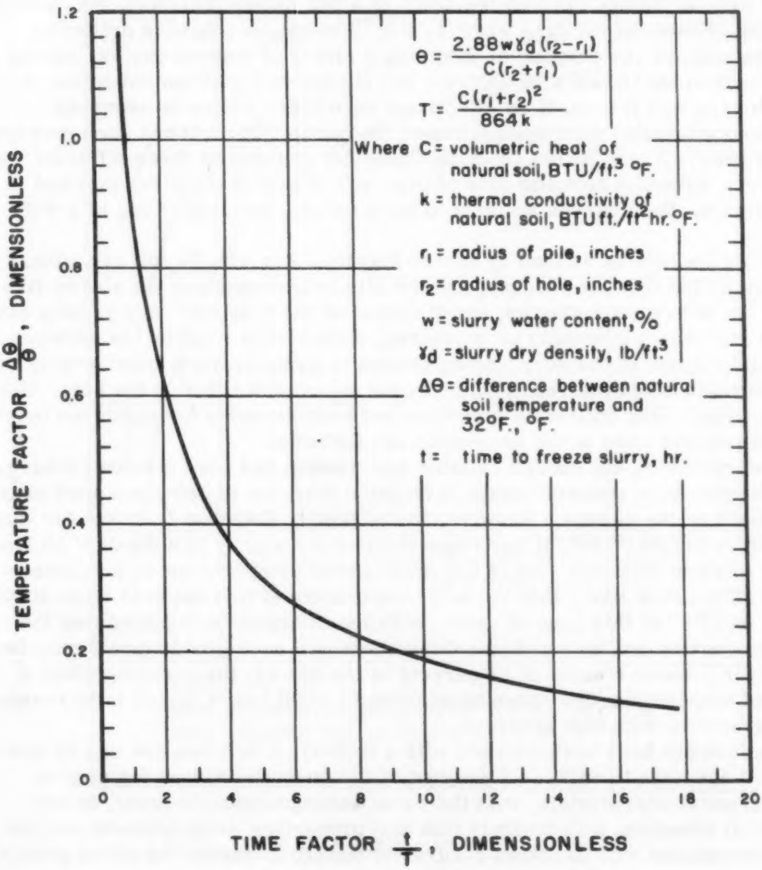

NATURAL
SOIL
SLURRY
PILE
1
2
r_1
r_2
Δr
Δr
Δr
Δr
Δr
PLAN
$5r_2$ (APPROX)
1
2
DASHED LINES SHOW
"LUMP" BOUNDARIES
PROFILE
TEMPERATURE, F
32°
INITIAL TEMPERATURE
DISTRIBUTION
DISTANCE
TEMPERATURE, F
32°
FINAL TEMPERATURE
DISTRIBUTION
DISTANCE

FIGURE 1



TIME REQUIRED TO FREEZE SLURRY AROUND A PILE IN PERMAFROST

FIGURE 2

pile and hole dimensions and the properties of the slurry, can be used to calculate a value on the y-axis. Entering the curve with this value gives a dimensionless number on the bottom scale from which the slurry freezing time at the depth considered can be computed, again using the soil properties and pile and hole diameters at this depth.

It may be pointed out that for holes of a few inches diameter, the curves will be of value in the case when $r_1 = 0$. Sometimes a hole is drilled in permafrost for the purpose of inserting a string of thermocouples, and the hole is then backfilled with slurry. The curves will give an indication of when this slurry will freeze, if an effectively zero inner radius is assumed.

As an example of the application of the curve, Figure 3 has been prepared using Figure 2, and shows freezing times for a slurry at three different water contents, around a common size of pile, in a soil with the given thermal properties. Such curves would hold for a natural soil consisting of a silty clay.

When the hole is formed by steam-thawing, it is usually full of a slurry caused by the thawing process, and the pile is inserted into the slurry-filled hole. In such circumstances, the diameter of the hole will vary greatly with depth and cannot generally be measured, except very roughly. In addition, the thermal regime of the surrounding ground is modified very considerably by the elevated temperatures attained by the steam and water in the hole. Use of the accompanying figures is therefore not recommended for instances in which steam jets are used in the excavation of pile holes.

Unfortunately, not enough reliable information has been obtained from actual temperature measurements in freezing slurries to provide a good practical check on the curves. However, in one test at Kotzebue (Alaska) Air Force Station in October 1955, it was observed that the slurry at a depth of 15 feet froze in about 15 hours. Since the mean annual temperature at Kotzebue is about 20°F., it is likely that the soil temperature at this depth is from about 23°F. to 25°F. at this time of year. With the information supplied that the soil properties and hole and pile dimensions are such that Figure 3 may be used, for a water content of 20 percent in the slurry, the curve supplies a time of freezing for this example of from 16 to 20 hours, which is in reasonable agreement with that observed.

The results have been checked with a numerical solution and can be considered accurate to within $\pm$ 5 percent of the correct solution for such a lumped parameter system, with the same assumptions. However, in any practical situation, it is unlikely that soil properties, temperatures and the hole dimensions will be known accurately enough to enable the above precision to be attained. Time values of $\pm$ 50 percent may well be encountered.

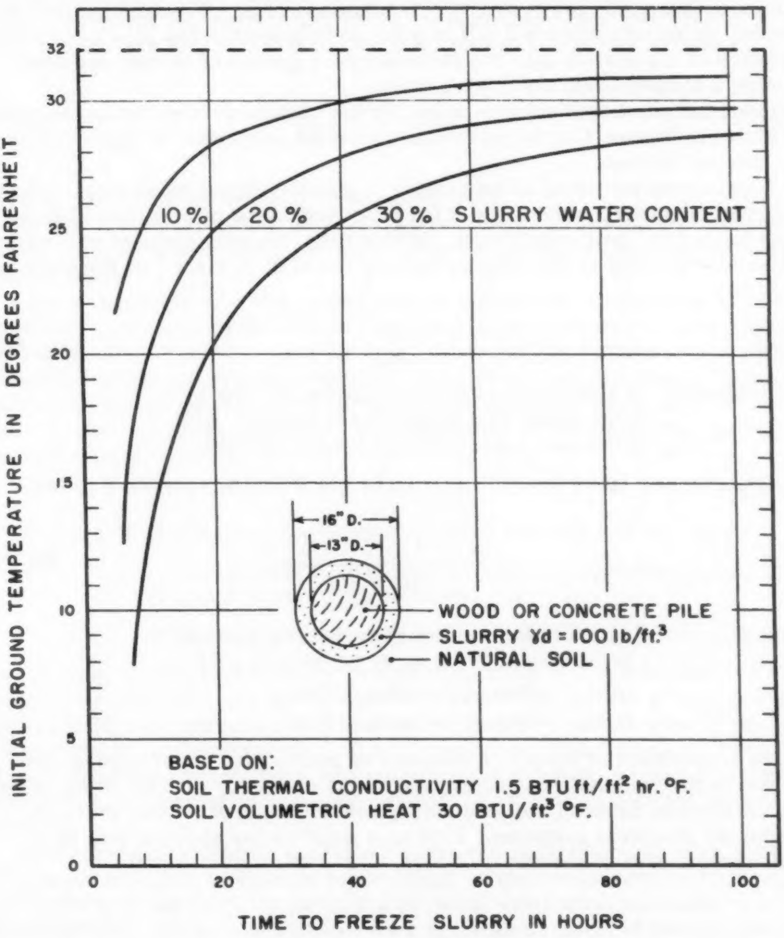
ACKNOWLEDGMENT

The assistance of E. F. Geary, Technical Aid, in the operation of the computer is acknowledged.

APPENDIX

Derivation of Dimensionless Parameters in Figures

In order to establish most conveniently the connection between the electrical model and the physical problem, each system was reduced to dimensionless



SPECIFIC SOLUTION OF SLURRY FREEZING PROBLEM

FIGURE 3

parameters. The electrical terms are of no interest to the present discussion except that they permit a solution to be plotted in terms of dimensionless coordinates indicating temperature difference and time. Then the scaling of the thermal problem resolves itself into the development of the physical characteristics of the system into two dimensionless groups of terms; one representing a temperature, the other a time.

Since the electrical solution holds for the particular distribution simulated as shown in Figure 1, it is necessary to use the properties of these lumps in the thermal system.

In problems involving latent heat, it is possible to establish a characteristic temperature based on the ratio of the total latent heat of a representative lump to its total heat capacitance. In this case, the only element assumed to possess latent heat is the lump containing the slurry, lump 1 in Figure 1.

Then

$$\theta = \frac{L}{C_s} \qquad (1)°F$$

where θ is a characteristic temperature of lump 1, °F
L is the latent heat of the slurry annulus, BTU
C_s is the heat capacitance of lump 1, BTU/°F

Assuming the latent heat of water to be 144 BTU/lb, equation 2 follows:

$$L = \frac{144 \cdot w \cdot \gamma_d \cdot (r_2^2 - r_1^2)}{100 \cdot 288} \qquad (2)\ \text{BTU}$$

where w is the water content of the slurry, percent
γ_d is the dry unit weight of the slurry, pcf
r_1 is the radius of the pile, inches
r_2 is the radius of the hole, inches

The capacitance of lump 1 is obtained by multiplying the volume of the element by the volumetric specific heat of the material, with the assumption that the specific heats of soil and slurry are not much different, which is true for all practical purposes. This also neglects the specific heat of a small central core of the pile.

$$C_s = \frac{C}{288} \cdot \frac{(r_1 + r_2)^2}{2} \qquad (3)\ \text{BTU/°F}$$

where C is the volumetric specific heat of the soil, BTU/ft^3°F

From equations 2 and 3, we get

$$\theta = \frac{2.88 \cdot w \cdot \gamma_d \cdot (r_2 - r_1)}{C(r_1 + r_2)} \qquad (1)a\ °F$$

By making the assumption that the slurry is initially unfrozen and at 32°F, a dimensionless temperature parameter is given by $F = \frac{\Delta\theta}{\theta}$ where $\Delta\theta$ is

the temperature difference between 32°F and the undisturbed temperature of the surrounding ground. Differing values of soil properties, the dimensions of the system and the surrounding ground temperature then give rise to a range of numbers for F, the dimensionless temperature characteristic.

Thus
$$F = \frac{\Delta\theta \cdot C \cdot (r_1 + r_2)}{2.88 \cdot w \cdot \gamma_d \cdot (r_1 - r_2)} \qquad (4)\,a$$

which is plotted as the vertical scale of Figure 2.

For the horizontal axis of Figure 2 a time characteristic of the thermal system must be sought along the lines of the temperature characteristic developed above.

A time characteristic of a system can be obtained by multiplying a representative capacitance of the system or a part of the system, by a representative resistance. In electrical apparatus, electrical capacitance and resistance are used; in thermal systems, thermal capacitance and resistance. When one system is used as an analogy to another, time characteristics must be obtained by the use of capacitances and resistances in corresponding parts of the two systems; in thermal terms

$$T = RC_s \qquad (5)\ \text{hours}$$

where T is the time characteristic of the thermal system, hours

R is the thermal resistance between the two chosen lumps (1 and 2 in Fig. 1) °F hr/BTU

C_s is the thermal capacitance of a chosen lump (here, 1) BTU/°F

The time characteristic in the present electrical model was based on the capacitance of lump 1 and the resistance between the centres of lumps 1 and 2. Consequently, the corresponding thermal properties must be used to develop the thermal time characteristic. It is of course possible to use the product of the capacitance of any lump and resistance of any path, provided, as stated above, that the corresponding lump and path is utilized in both nature and the model.

In the thermal prototype equation 3 gives the capacitance of lump 1. The resistance of a path is directly proportional to the cross-sectional area and conductivity of the path. In problems of radial symmetry, the mean cross-sectional area of a path is obtained as a logarithmic mean of the two bounding circumferential areas, but to accuracies adequate for the present solution, it can be given as an arithmetic average of the end areas.

$$R_{1-2} = \frac{2}{3k}$$

where k is the thermal conductivity of the soil, BTUft/ft^{2}hr°F

Then
$$T = \frac{2}{3k} \cdot \frac{C}{288} \cdot \frac{(r_1 + r_2)^2}{2}$$

$$= \frac{C(r_1 + r_2)^2}{864k} \qquad (5)\,a\ \text{hours}$$

The dimensionless time parameter is then given by

$$T_d = \frac{t}{T} \qquad \text{(6) dimensionless}$$

where t is the time taken by the slurry to freeze, hours.

Values of T_d also depend on the soil characteristics and the dimensions of the system, since

$$T_d = \frac{864kt}{C(r_1 + r_2)^2} \qquad \text{(6) a, dimensionless}$$

which is plotted on the horizontal scale.

Both equations (4) a and (6) a contain numerical factors which could have been incorporated into the numbers on the general solution Figure 2, but it was felt that a clearer understanding of the problem would be obtained by retaining them in the parameters. This has the additional advantage of making the ordinates convenient whole numbers.

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2. SCOTT, R. F., "An Hydraulic Analog Computer for Studying Diffusion Problems in Soil." Geotechnique, VII, 2, 55-72, June 1957.

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Proceedings of the American Society of Civil Engineers

COMPRESSIBILITY AS THE BASIS FOR SOIL BEARING VALUE

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ABSTRACT

The value and practicability of rupture theory as the chief basis for bearing value determination is questioned. As an alternative it is proposed that contact pressures and footing sizes be selected so as to equalize settlement due to soil compression. Procedures for use in practical applications are proposed.

INTRODUCTION

Spread foundations are intended to distribute concentrated wall or column loads so that settlement from any cause is limited to some specified minimum value. This is accomplished in practice by proportioning the foundation element so that the contact pressure does not exceed what is termed the allowable bearing value of the soil. Current methods for determining allowable bearing values differ widely in nature from the crudest, empirical procedures to various forms of engineering analysis. The cruder methods lead to designs ranging from the overconservative to the inadequate. Present-day analytical procedures require special programs of subsurface investigation, laboratory testing and engineering studies which are impractical in routine site investigations and may lead to designs which are inconsistent with the objective of minimizing differential settlement. Thus, it is believed that there are grounds for a critical review of the subject and for presenting suggestions for improvement of present techniques.

Load-settlement relations for a spread foundation are represented in generalized form in Fig. 1(a). Under relatively low values of contact pressure, in the range from 0 to point a, it is seen that settlement is roughly

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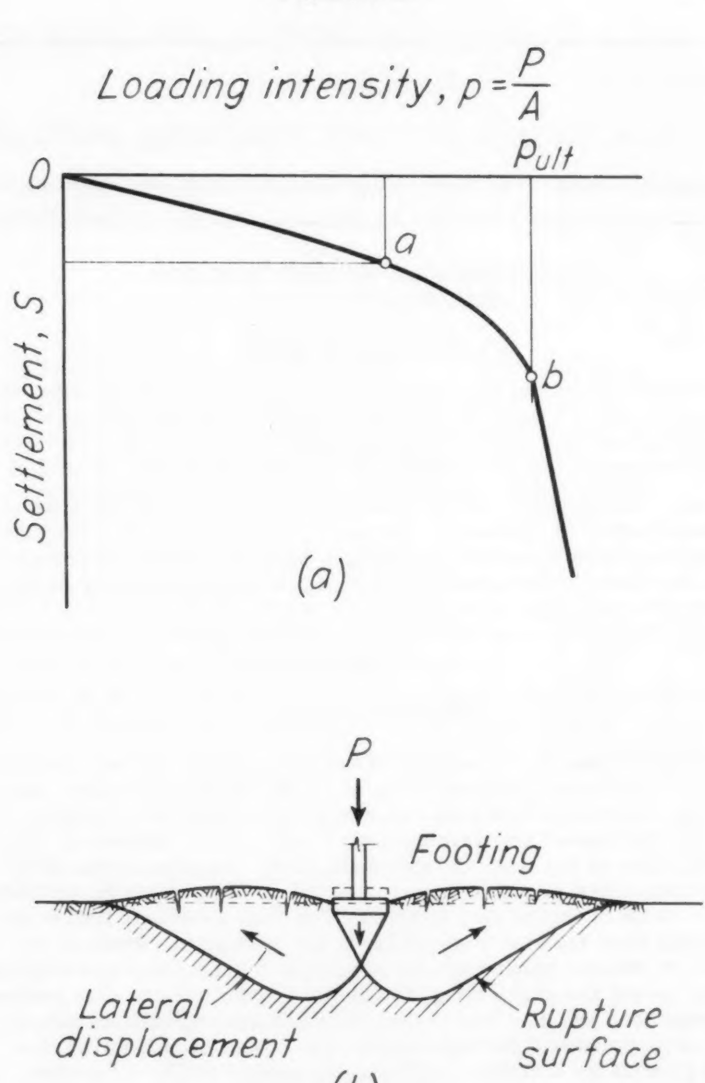


LOAD-SETTLEMENT RELATIONS, SPREAD FOOTINGS

FIG. 1

proportional to loading. In this range, it is often considered that settlement is due chiefly to one dimensional soil compression. At higher values of loading, a point is reached such as at b, at which the soil is ruptured* and gross settlement occurs as a result of lateral displacement of the soil as illustrated in Fig. 1(b). Thus, both soil compressibility and shearing strength may require consideration in determining allowable bearing value.

Many present-day analytical procedures for determination of soil bearing value require evaluation of the shearing strength and ultimate bearing value of the supporting material as an initial step. The allowable bearing value or limiting contact pressure for use in design is then frequently determined as an arbitrary fraction of the ultimate value. This procedure in itself involves no analysis of soil compression under the design loading. An analysis of settlement due to soil compression, if undertaken at all, is often regarded as a secondary rather than primary operation and one which is of interest chiefly with clays. As a result, there has been a growing tendency to consider that allowable soil bearing value is related principally to shearing strength and that factors which affect strength, affect bearing value in like degree.

To justify the above concept, it would be necessary to show that there is a consistent relationship between the ultimate bearing value and the loading which produces no more than a specified amount of settlement. It is difficult to demonstrate that such a relationship exists. Certain soils, uniform sand, for example, are extremely incompressible, yet relatively weak in shear (low friction angle) while with others, such as partially saturated, well graded cohesionless or mixed soils, the opposite may be the case. Beside soil characteristics, such factors as footing size and depth below the ground surface and the extent and degree of surcharge, affect ultimate and allowable bearing values in differing degree. As a result, if foundation design is based on providing a constant factor of safety against soil rupture, it is almost certain that under the design loading, adjacent footings will settle differentially. For this and other reasons, it appears that soil shearing strength alone is not a satisfactory index of allowable bearing capacity.

Assuming that at least a minimum safety factor against rupture is provided in all cases, and that this can be accomplished with little more than perfunctory consideration, it is believed that selection of allowable bearing values should be based primarily on considerations of soil compressibility as affected by soil and foundation characteristics. Should this belief gain sufficient support to justify its adoption generally, there would be a rather far-reaching change in present thinking and procedures. There would be a lessening of interest in the manifold attempts now being made to correlate bearing value with shearing strength and consequently some de-emphasis on methods for measuring strength both in the field and in the laboratory. There would, of course, be a corresponding increase of interest in the development of procedures for determining the field compression characteristics of natural soil formations.

In view of the above considerations, an examination has been made of the possibility of developing sound, yet practical methods for determining allowable bearing value chiefly with reference to soil compressibility. For this purpose, it has been found advantageous to modify some of the existing relationships between stress and soil compression and to originate new analytical

*The loading intensity at the point of soil rupture is known as the ultimate bearing value of the soil, P_{ult}.

procedures. An account of these developments and conclusions based thereon, is given in the following discussion.

Relations Between Contact Pressure and Settlement Due to Soil Compression

Compression of Layers of Finite Thickness

Soil compressibility is conventionally expressed in terms of the change in void ratio Δe produced by a given change in loading. When compressibility data are presented in semi-logarithmic form as shown in Fig. 2(a), there are a number of situations in which the compression diagram is approximately linear at least over a limited range of loading. The slope* of straight line sections of these semi-log diagrams is termed the compression index of the soil and is designated C_c. Evidently,

$$C_c = \frac{\Delta e}{\Delta \log p}$$

or,

$$\Delta e = C_c \log \left(1 + \frac{\Delta p}{p_i} \right) \tag{1}$$

in which

p_i = initial vertical stress under which an element of soil at given depth has been consolidated prior to application of a boundary loading which creates the stress increment, Δp.

In studying soil compressibility and its role in settlement of structures, engineers have devoted primary consideration to one-dimensional compression.** The change in thickness of soil elements due to such compression is related to change in void ratio in conventional procedures by the expression

$$\Delta H = H \frac{\Delta e}{1 + e} \tag{2}$$

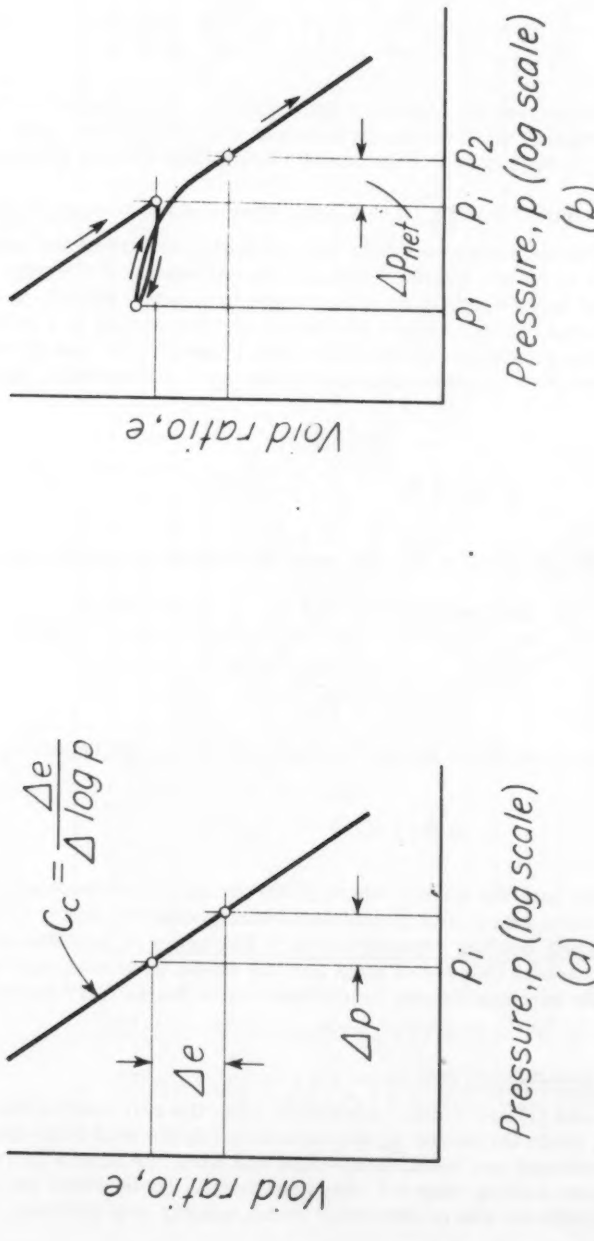
in which,

H = initial thickness of compressible material

e = initial void ratio

*The slope is negative according to standard conventions but this is usually disregarded since it is only the numerical value of the index which is of practical interest in engineering applications.

**This is justified by the assumption that when contact pressures on foundations have been appropriately limited, shearing stresses are relatively small and lateral strains are negligible.



COMPRESSION DIAGRAMS

FIG. 2

The two foregoing equations may be combined to obtain the familiar expression

$$\Delta H = H \frac{C_c}{1+e} \log\left(1 + \frac{\Delta p}{p_i}\right) \tag{3}$$

Equation (3) indicates that the change in thickness of a soil element or layer due to one-dimensional, vertical compression is a function of the ratio of the stress increment to the initial stress on the element and that the proportionality factor is the quantity $\frac{C_c}{1+e}$ a term known as the compression ratio which involves both the compressibility and the initial density of the soil.

Equation (3) is in a form which is suitable for calculating the change in thickness of a soil layer due to a specific change in stress. When it is desired to calculate the extent to which the initial vertical stress in a soil interval may be changed without producing more than a specific amount of compression, it is desirable to solve equation (3) for Δp. The solution may be written,

$$\Delta p = \left(10^{\frac{\Delta H}{H}\frac{(1+e)}{C_c}} - 1\right) p_i \tag{4}$$

Some simplification of equation (4) may be accomplished by introducing the terms,

$$\text{Settlement,} \quad S = \Delta H \tag{5}$$

and

$$C = \frac{1+e}{C_c} \tag{6}$$

After substitution of the above quantities, equation (4) for Δp becomes,

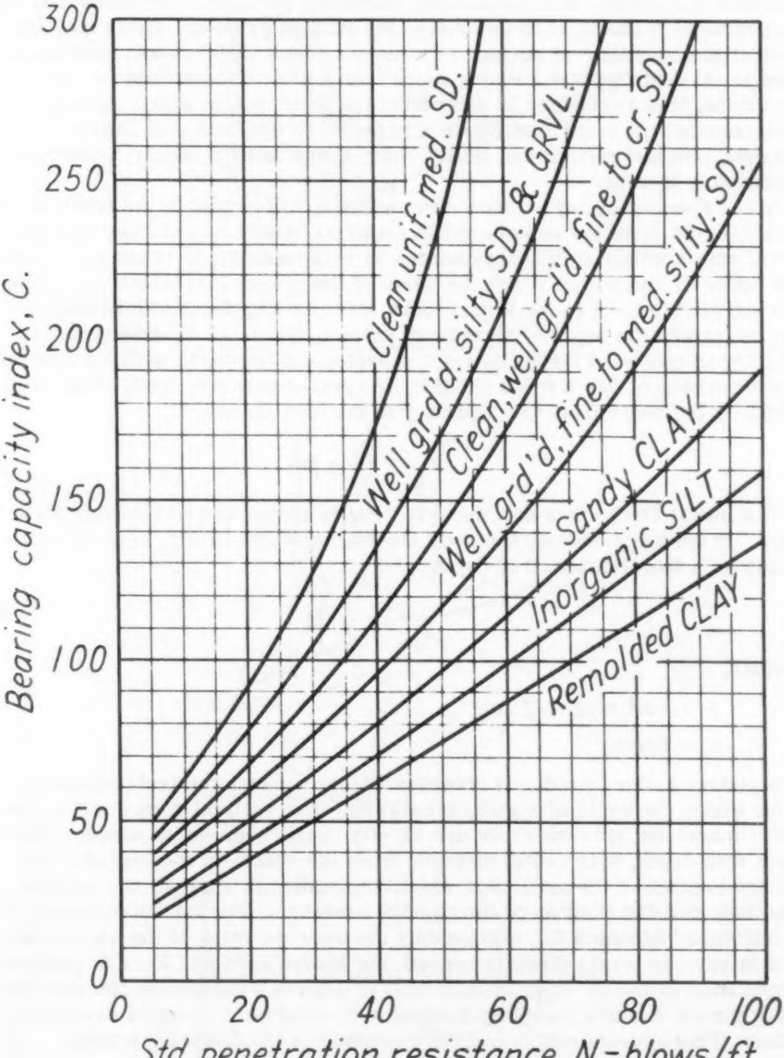
$$\Delta p = \left(10^{\frac{SC}{H}} - 1\right) p_i \tag{7}$$

It may be shown that the term C which is the reciprocal of the compression ratio, is to some extent an index of soil bearing capacity and it is accordingly designated, bearing capacity index.* The order of magnitude of index values for various classes of soils and the extent to which these values are affected by the in-place density or consistency of the soil are shown in Fig. 3.

Compression of Unstratified Soil

Equations (3) and (7) are readily applicable when the soil interval has a relatively small, finite thickness H, throughout which the void ratio and initial stress conditions are sensibly constant and when the stress increment produced by a given loading does not vary greatly with depth within the soil interval. Such conditions are occasionally found, notably in a confined,

*It may be noted that this is a dimensionless term.



BEARING CAPACITY INDEX VALUES

FIG. 3

consolidation test specimen or in a relatively thin substratum of compressible material in a natural soil formation. However, perhaps more often than not, the practical problem is to determine the contact pressure which may be applied at the boundary or surface of a more or less uniform soil formation of substantial depth without causing more than a specified settlement. For such conditions, it is necessary to determine the depth within which significant compression will occur and within this depth, to evaluate soil characteristics and establish the variation of initial body stress and the stress increment due to boundary loading.

In the absence of any stratification which would establish the position and the thickness $\underline{H}$ of the compressible material, it will be assumed that the interval within which significant changes in void ratio due to boundary loading will occur is that within which the value of the stress increment $\underline{\Delta p}$ is significant with respect to the initial, body stress. The depth of this interval will be termed the depth of significant stress. This will be arbitrarily defined for purposes of the following discussion, as the depth within which the stress increment has a value equal to or greater than one-tenth of the body stress or the depth from the boundary to the level at which,

$$p_i = 10\Delta p \tag{8}$$

The initial stress in a soil mass is usually stress due to the body force alone. In unstratified soil, the body stress, as shown in Fig. 4(a) varies simply as a linear function of depth, or

$$p_i = \gamma h \tag{9}$$

in which,

γ = unit weight of soil

h = depth

Boundary loading produces stresses which increase the body stress at all points within the depth of significant stress. The vertical component of the stress increment with which we are chiefly concerned, varies among other things with depth, with radial distance from the center of the loaded area, with the breadth of the area over which the loading is applied, and with the magnitude and distribution of the contact pressure. For this discussion, it is considered adequate to evaluate only the average value of the stress increment at various levels directly beneath the loaded area and for this purpose a modification of the 60° approximation(1) is used in establishing the distribution of stress due to a boundary loading of limited lateral extent such as a footing. This approximation can be expressed in the following terms,

$$\Delta p = \frac{B^2}{(h+B)^2}\, p_c \tag{10}$$

in which,

Δp = average value of stress increment at depth $\underline{h}$ over an area defined by planes at 63-1/2° with the horizontal descending from the edges of the area over which the boundary loading is applied.

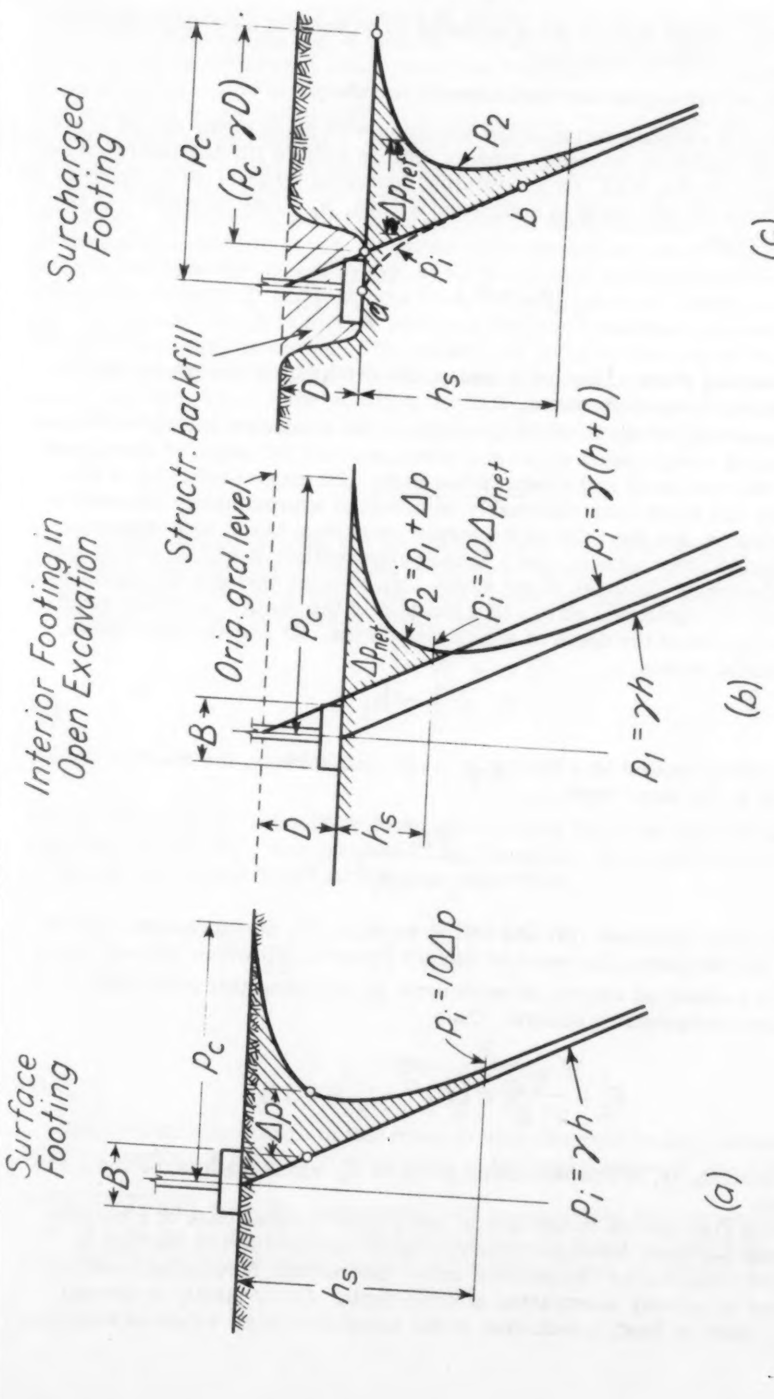

Surface Footing
Interior Footing in Open Excavation
Surcharged Footing
Structr. backfill
Orig. grd. level
(a)
(b)
(c)

FIG. 4

B = width of area (area assumed to be square) over which boundary loading is applied.

P_c = average contact pressure at boundary.

A graphical representation of the variation with depth of the stress increment due to a uniform boundary loading over an area of limited lateral extent is also given in Fig. 4(a). By substituting equations (9) and (10) in equation (8), an expression for the depth of significant stress, $\underline{h_s}$, may be written in the following terms.

$$h_s (h_s + B)^2 = \frac{10 B^2}{\gamma} P_c \tag{11}$$

Note that for given values of $\underline{B}$ and $\underline{\gamma}$, the depth of significant stress is a function of the contact pressure, $\overline{P_c}$.

Settlement or deflection of the boundary under moderate loading will occur as a result of compression of the soil throughout the full depth of significant stress. The void ratio and compression index (not compressibility) of the soil within this depth may reasonably be assumed approximately constant in many instances, but the ratio of the stress increment to the body stress obviously varies considerably. As a first approximation, it will be considered that the change in thickness of the entire soil interval having a thickness equal to the depth of significant stress is a function of the change in stress which occurs at one-third the depth of significant stress. At one-third the depth, $\underline{h_s}$, the initial stress,

$$p_i = \frac{1}{3} \gamma h_s \tag{12}$$

and for loading applied by a footing at the ground surface, the average stress increment at the same depth is,

$$\Delta p = \frac{9 B^2}{(h_s + 3B)^2} P_c \tag{13}$$

By substituting equations (12) and (13) in equation (7), an expression may be obtained for the particular value of contact pressure $\underline{p'_c}$ which will not cause more than a specified amount of settlement $\underline{S}$, providing that only one-dimensional compression occurs. Thus

$$p'_c = \frac{\gamma}{27 B^2} H \left(10^{\frac{SC}{H}} - 1\right)(H + 3B)^2 \tag{14}$$

In this equation, $\underline{H}$, is the particular value of $\underline{h_s}$ which applies for the given conditions.

Equation (14) applies to the special and rather unusual case of a surface loading and has been developed chiefly to give an indication of the type of expression required for the purpose under discussion. Structural foundations are almost invariably constructed at some depth. Consequently in normal practice, there is first, a reduction in the initial stress as weight of overlying

soil is removed and then an increase in stress as the foundation and loading due to structural backfilling is applied. These are factors which may significantly affect settlement under moderate loading and which, therefore, deserve consideration. However, the effects of excavation and footing construction at depth are somewhat difficult to generalize. The manner and degree of stress reduction, for example, depends on the depth and lateral extent of the excavation. The stress increase is affected by the position of the foundation element within the excavation area and by the surcharge or structural backfill, if any. Analysis of the effect of excavation must, therefore, be made with reference to a specific set of conditions. The conditions which will be considered first are illustrated in Fig. 4(b). Essentials are that the excavation is of uniform depth D and that it is of considerable lateral extent in comparison with this depth. It is further assumed that the foundation element is to be located at or near the center of the excavation and that there is no backfill.

For the given conditions, it is convenient to measure depth h from the bottom of the excavation. The equation for initial body stress in the soil beneath the floor of the excavation in this case would therefore be written,

$$p_i = \gamma(h + D) \tag{15}$$

For the assumed conditions, the stress reduction due to excavation may be considered approximately constant with depth* within the depth of significant stress. Thus, after excavation, the body stress beneath the center of the building area for a considerable depth, would be

$$p_1 = p_i - \gamma D$$

or,

$$p_1 = \gamma h \tag{16}$$

The distribution of stress after foundation loading has been applied (also illustrated in Fig. 4(b), may be established by adding the stress increment due to foundation loading to the stress after excavation,

$$p_2 = p_1 + \Delta p \tag{17}$$

or in this case,

$$p_2 = \gamma h + \frac{B^2}{(h+B)^2} p_c \tag{18}$$

Compression or change in thickness of soil intervals under increased vertical loading is related to the change in the initial value of the void ratio,

---

*This holds only when the excavation is of considerable lateral extent compared with its depth, as originally postulated. It would not apply for example, to a localized excavation for an individual wall or column footing.

as previously stated. When loading is first reduced and then subsequently restored, void ratio as shown in Fig. 2(b), is not changed appreciably. Thus, so far as settlement is concerned, it is only the net change in stress or difference between the initial and final stress which is of practical interest in most cases. The net stress increment at any depth beneath the center of the area would be

$$\Delta p_{net} = p_2 - p_i \tag{19}$$

or in this case,

$$\Delta p_{net} = \frac{B^2}{(h+B)^2}\, p_c - \gamma D \tag{20}$$

The depth of significant stress for this case will be taken as the depth from the floor of the excavation to the point at which the net stress increment is one-tenth the initial body stress. At this depth,

$$p_i = 10\, \Delta p_{net} \tag{21}$$

Substitution of equations (15) and (20) in equation (21) results in the following expression for depth of significant stress,

$$(h_s + 11D)(h_s + B)^2 = \frac{10B^2}{\gamma}\, p_c \tag{22}$$

At one-third the depth of significant stress, the initial stress has the value,

$$p_i = \frac{\gamma}{3}(h_s + 3D) \tag{23}$$

and the net stress increment,

$$\Delta p_{net} = \frac{9B^2}{(h_s+3B)^2}\, p_c - \gamma D \tag{24}$$

Equations (23) and (24) may now be substituted in equation (7) to obtain an expression for the particular value of the contact pressure p'_c which can be applied without causing more than a specified settlement S, viz,

$$p'_c = \frac{\gamma}{27B^2}\left[(H+3D)\,10^{\frac{Sc}{H}} - H\right](H+3B)^2 \tag{25}$$

It will be noted that if the value D = 0 is substituted in equations (22) and (25), equations (11) and (14) which were developed for the surface footing are obtained.

It is now desired to develop basic relationships for one more typical foundation condition. This condition is that of a surcharged footing. A surcharged footing is one which is constructed in an excavation usually of limited

extent laterally and then backfilled on one or more sides. An illustration of a footing which has been completely backfilled is shown in Fig. 4(c).* By methods similar to those used previously, the following equations may be developed for this case.

As in the previous case, it is convenient to express initial stress in terms of footing depth and depth beneath the footing, viz,

$$p_i = \gamma(h + D) \tag{15}$$

Excavation reduces the initial stress as before. It is difficult to write an expression for the magnitude and variation of the stress reduction unless the exact dimensions of the excavation are known. However, it may be said that when the excavation is of limited extent, the stress reduction is significantly less than in the previous case as shown by the qualitative indication of the variation of stress $\underline{p_1}$ after excavation (line $\underline{ab}$) in Fig. 4(c). An expression for stress $\underline{p_2}$ after loading is applied by nevertheless be written in the form,

$$p_2 = p_i + \Delta p_{net} \tag{26}$$

The net stress increment in this case, is a function of the net contact pressure. When the difference between unit weight of backfill and unit weight of concrete is small, the net contact pressure may be expressed simply as $p_c - \gamma D$, it being understood that $\underline{p_c}$ reflects column loading plus weight of foundation element plus backfill as is customary. Thus,

$$\Delta p_{net} = \frac{B^2}{(h + B)^2}\left(p_c - \gamma D\right) \tag{27}$$

Note the difference between equations (27) and (24). For this case, an expression for the depth of significant stress would be obtained by substituting equations (15) and (27) in equation (8).

$$10\frac{B^2}{(h_s + B)^2}\left(p_c - \gamma D\right) = \gamma(h_s + D)$$

or to obtain a form suitable for comparison with equation, (22),

$$(h_s + 11D)(h_s + B)^2 - 10Dh_s(h_s + 2B) = \frac{10B^2}{\gamma}p_c \tag{28}$$

As before, the initial stress at one-third the depth $\underline{h_s}$, has the value,

$$p_i = \frac{\gamma}{3}(h_s + 3D) \tag{23}$$

In this case however, the net stress increment at one third the depth h_s is,

*Figs. 4(a), 4(b), and 4(c) are all drawn to the same scale to permit comparison.

$$\Delta p_{net} = \frac{9B^2}{(h_s+3B)^2}(p_c - \gamma D) \tag{29}$$

a value which is seen to differ from that given in equation (24) for the footing constructed without surcharge, as is to be expected. The expression for the particular value $\underline{p'_c}$ of the contact pressure which will cause no more than a specified settlement for this case is obtained by substituting equations (23) and (29) in equation (7),

$$\frac{9B^2}{(H+3B)^2}(p'_c - \gamma D) = \frac{\gamma}{3}\left(10^{\frac{SC}{H}} - 1\right)(H+3D)$$

or to obtain a form suitable for comparison with equation (25),

$$p'_c = \frac{\gamma}{27B^2}\left[(H+3D)10^{\frac{SC}{H}} - H\right](H+3B)^2 - \frac{\gamma}{9B^2}DH(H+6B) \tag{30}$$

While there are many other foundation conditions of practical interest, the two described above, namely foundations with and without surcharge plus the special case of the surface foundation will suffice for purposes of this discussion. Methods for obtaining solutions to the equations for depth of significant stress and limiting contact pressure will now be described.

As previously stated, the depth of significant stress $\underline{h_s}$, for given foundation conditions is a function of the contact pressure. For the contact pressure which will cause a specific amount of settlement the depth of significant stress has the particular value, $\underline{H}$ which appears in equations for limiting contact pressure. Substituting this value $\underline{H}$ in the general equations for depth of significant stress, it is possible to write simultaneous equations from which the value of $\underline{H}$ may be determined. Those for the case of the footing near the center of a laterally extensive excavation are given below.*

From equation (22),

$$(H+11D)(H+B)^2 = \frac{10B^2}{\gamma}p'_c = K_1 \tag{31}$$

and from equation 25,

$$\frac{10}{27}\left[(H+3D)10^{\frac{SC}{H}} - H\right](H+3B)^2 = \frac{10B^2}{\gamma}p'_c = K_2 \tag{32}$$

The value of H may then be determined by trial as the value for which,

$$K_1 = K_2 \tag{33}$$

*Equations in the same form may also be obtained for the surcharged footing.

When $\underline{H}$ has been determined, values can be obtained for a term $\underline{K'}$ defined simply as,

$$K' = K_1 = K_2 \tag{34}$$

Evidently,

$$K' = \frac{10B^2}{\gamma} p'_c \tag{35}$$

For purposes of generalization, it is now desirable to introduce the term

$$K = \frac{K'}{10B^2} \tag{36}$$

whence,

$$p'_c = \gamma K \tag{37}$$

The term $\underline{K}$ in equation (37) is a useful parameter of allowable contact pressure which is referred to hereinafter as the allowable pressure index.* It is evident that $\underline{K}$ is a function of footing width $\underline{B}$ and depth $\underline{D}$, tolerable settlement $\underline{S}$ and of soil compressibility as expressed by the bearing capacity index $\underline{C}$. The interrelationship of these variables for a given value of D is shown in Fig. 5 and is discussed in subsequent sections of this paper. It will be noted that in the relationship between $\underline{K}$ and the allowable contact pressure p'_c given by equation (37), the unit weight of soil γ, is an independent variable.

Discussion of Results

Although it has been stated as a general proposition that depth of significant stress beneath a footing of given size varies with contact pressure, it is now apparent that when contact pressure is to be limited so as to cause only a given settlement, the depth of significant stress depends only on soil compressibility and the tolerable settlement. Thus, the depth of significant stress does not appear explicitly in Fig. 5, and is not required in determining $\underline{K}$, providing diagrams like Fig. 5 are available. Nevertheless, the value of the depth of significant stress and its variation with various foundation conditions is of considerable practical interest.** In this connection, the following comments can be made.

---

*This term has the dimensions of length, for which reason, when γ is in pounds per cubic foot as is customary, $\underline{K}$ and the dimensions $\underline{H}$, $\underline{D}$, $\underline{B}$, and $\underline{S}$ must all be in feet.

**Depth of significant stress is required in planning a sub-surface investigation, it being important to have at least one boring which descends to this depth or a little beyond. It is generally justifiable to conclude that a weak layer will have little or no effect if it lies beneath the depth of significant stress. This depth is also a factor in the extent to which overlap of stress beneath adjacent footings may develop.

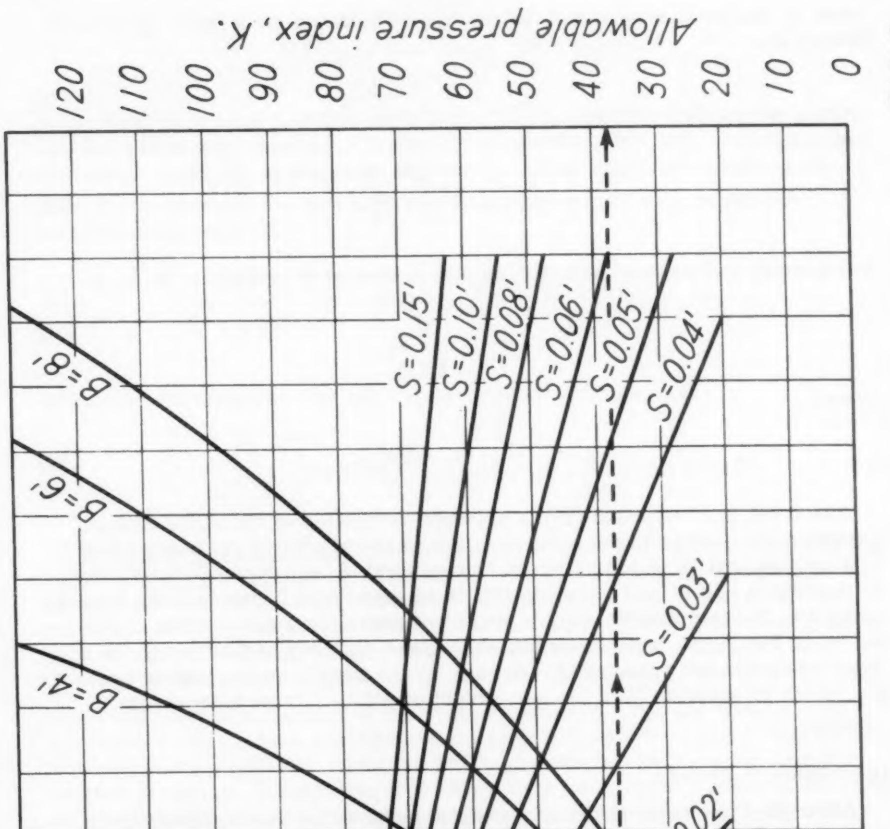

Allowable pressure index, K.
120
110
100
90
80
70
60
50
40
30
20
10
0
S=0.15'
S=0.10'
S=0.08'
S=0.06'
S=0.05'
S=0.04'
S=0.03'
S=0.02'
B=8'
B=6'
B=4'
B=2'
INTERIOR FOOTING
NO SURCHARGE
D=4 FT.
0
50
100
150
200
250
300
350
Bearing capacity index, C.

FIG 5

Utilizing results obtained from solutions of equations (31) and (32), it can be shown that for footings at a given depth, the depth of significant stress increases appreciably with soil compressibility and with the tolerable settlement allowance and in a somewhat lesser degree with footing width.* This is illustrated in Fig. 6. It can also be shown that in soil of a given compressibility the depth of significant stress, measured from the bottom of the footing, decreases with the depth of the footing below the ground surface. However, there is some evidence that the total depth from the ground surface to the point at which the stress increment is no longer significant is approximately a constant for a footing of given size and loading.

It may also be stated as a general proposition that under the definition embodied in equation (8) the depth of significant stress for a given loading is inversely proportional to unit soil weight, thus being substantially greater in submerged than in partially saturated soil. However, when the contact pressure is to be limited so that only a given settlement occurs, it now appears that depth of significant stress is independent of soil weight. For this reason, the position of the ground water table need not be known in evaluating the depth of significant stress for the conditions under discussion whereas it is a vital factor in determining the allowable contact pressure.

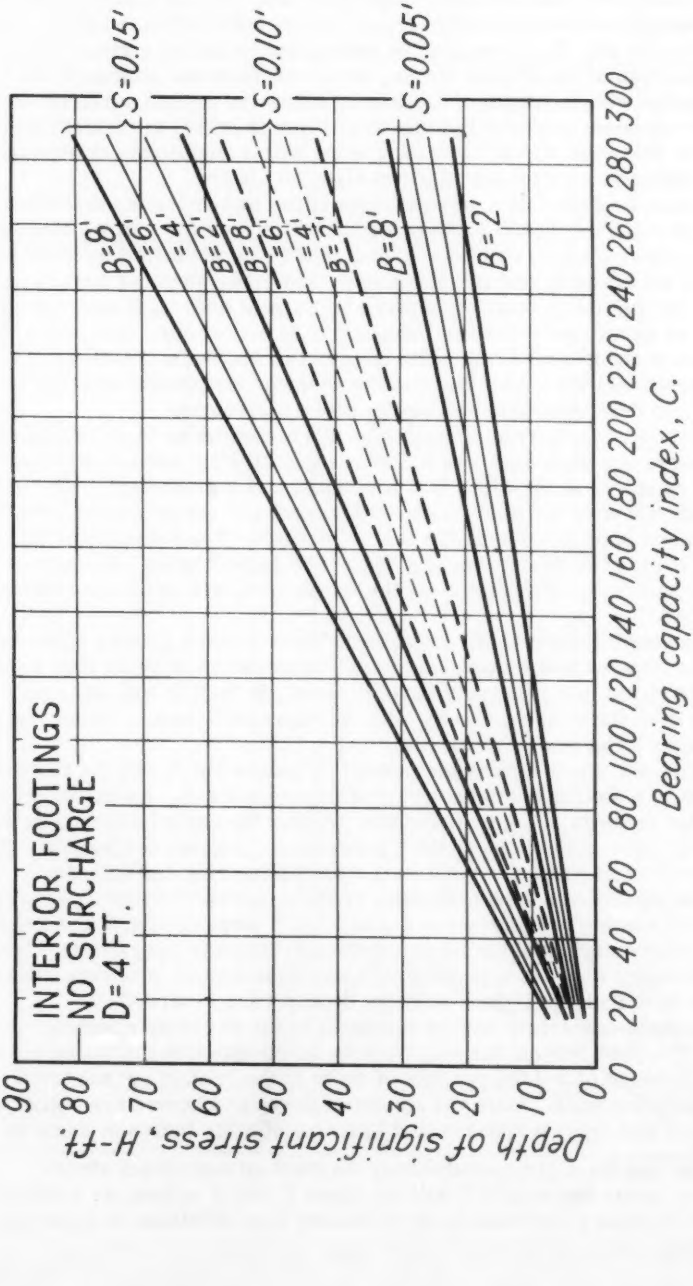
Implications of the foregoing equations and diagrams as to allowable contact pressures are discussed below. For a specified amount of settlement, it is evident as shown in Fig. 7(a), that for footings at a given depth, the contact pressure should vary inversely with footing size, i.e. lower contact pressures should be used in proportioning the larger footings. The converse of this statement is also illustrated, namely that if the same contact pressure is allowed with footings of different size, the larger ones will settle more than the smaller ones.

Fig. 7(a) also indicates that "size effects" are of much greater importance when the anticipated settlement due to soil compression is large than when it is to be restricted to a relatively small amount. In fact, it will later be shown that in certain cases, chiefly with incompressible soils, consideration of size effects is somewhat academic.

Another factor which affects the contact pressure which may be allowed, is the depth of a footing below the original ground surface. As shown in Fig. 7(b), for footings of a given size, the greater the depth below the original grade, the greater may be the contact pressure for a given settlement. This "depth effect" varies in importance both with footing size and with depth itself. This variation is such that there might be a considerable difference in the allowable contact pressures for a small and a large footing if the smaller is at appreciable depth and the larger at shallow depth (a very common situation, incidentally) whereas approximately the same contact pressure might be allowed for both footings if their relative depths were reversed.

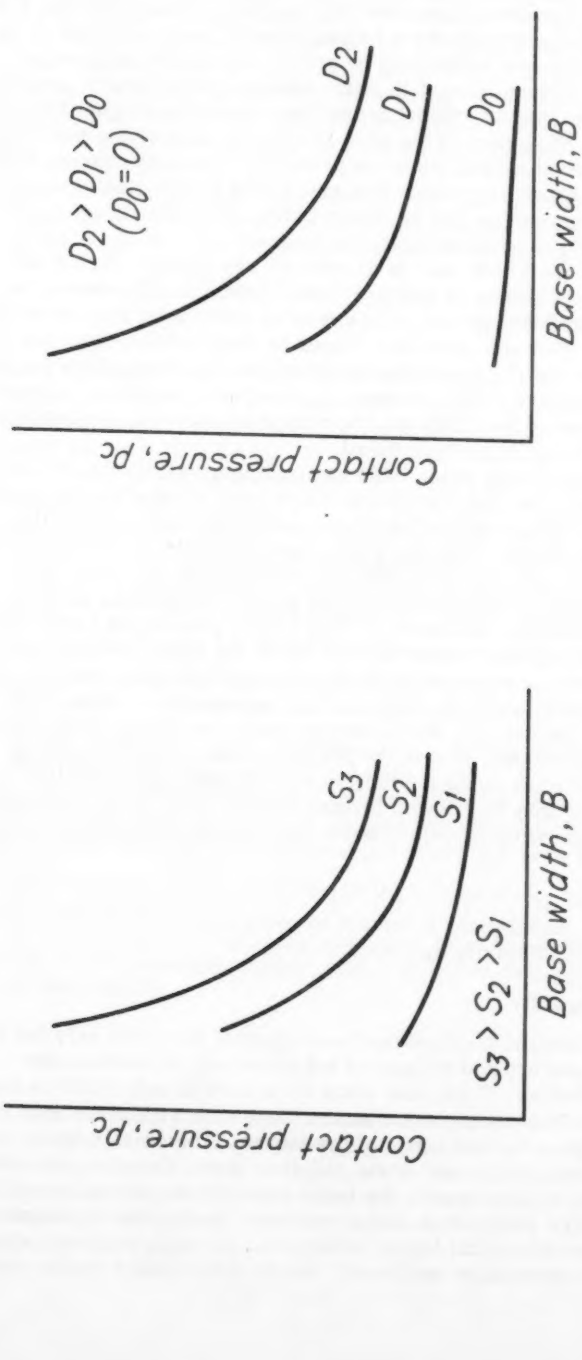
Adjustment of contact pressures for depth effect is rarely attempted in practice. The 1944 Boston Building Code is an exception in providing for an arbitrary increase of 2-1/2% per foot of depth in the contact pressures allowed on granular soils. Since the equations developed herein are entirely general as to soil type, it appears that this or a similar increase might be

*The finding that for a given settlement, the depth of significant stress varies as a linear function of $\underline{C}$ and for given $\underline{C}$ and $\underline{S}$ values, as a linear function of B, is of great assistance in making trial solutions of equations (31) and (32).



DEPTH OF SIGNIFICANT STRESS

FIG. 6



Footings at given depth, different settlement allowance.
(a)

Footings at different depths, given settlement allowance.
(b)

FIG. 7

allowed with soils of any type.

Without constructing additional diagrams, one more conclusion may be reached, simply by comparing equations (25) and (30). This is that for a given settlement, the contact pressure for a footing of given size and depth in an open excavation without surcharge may be greater than for a surcharged footing. There will be even more difference between the allowable column loads in the two cases than in contact pressures, wince the weight of the back-fill must be considered as part of the total load in the case of the surcharged footing. This conclusion as well as those preceding it are, of course, valid only for pressures of such magnitude that settlement is due chiefly to one-dimensional soil compression, but as shown below, it is this range of pressures which is usually of greatest practical interest.

The contact pressure which may be allowed on any footing, varies considerably with selected values of tolerable settlement. A difference of no more than 1/4 inch in assumptions as to tolerable settlement will more than double the indicated allowable pressure values in many cases. This can be seen by reference to Fig. 5. Since the relationships which have thus far been developed are so sensitive in this respect, assumptions regarding tolerable settlement become one of the chief considerations in practical applications, as is subsequently explained in greater detail.

The position of the ground water table has an important bearing on unit soil weight. It is also the case that formations which have always been submerged are in general more compressible than those above the water table. Unit soil weight does not vary widely in partially saturated formations. However, the difference between the unit weight of soil above and below the water table is appreciable, the former being about twice the latter. In applying equation (37), it would often be justifiable to assume a value of 120 pounds per cubic foot, for example, when the ground water table is below the depth of significant stress. However, with the ground water at or close to footing grade, a value of about 60 to 65 pounds per cubic foot would be appropriate. Thus, it is evident that for a given settlement, the allowable contact pressure in the latter case should be about one-half that in the previous case. If there is an appreciable difference in the compressibility of submerged and partially saturated formations, this will have a further effect. Few, if any, building codes require consideration of ground water elevation in establishing allowable bearing values.

Limitation of Contact Pressure for Security Against Soil Rupture

Ultimate Bearing Capacity

The above equations and conclusions based thereon are valid only for conditions under which the applied pressures act primarily to produce one-dimensional compression. Thus, they could be applied to soil which is rigidly confined laterally as in a compression testing device or with unconfined soil, they could be used for a limited range of pressures. For certain soils, considerations of tolerable settlement alone will lead to the limitation of contact pressures necessary to comply with the basic assumption. These would be the more compressible soils which under moderate loading would compress significantly. An unconfined but highly incompressible soil, however, would fail in shear before appreciable settlement due to compression could occur.

Thus, for soils of the latter type, settlement must be abandoned as a basis for pressure limitation and the provision of at least a minimum factor of safety against soil rupture adopted instead.

As previously explained, shear failure or rupture and lateral displacement of the soil beneath a spread foundation occurs when the contact pressure reaches what is known as the ultimate bearing value of the soil. Security against rupture is therefore obtained by estimating the ultimate bearing capacity of a given soil and then limiting contact pressures to some arbitrary fraction of this value.

One of the common procedures for determining ultimate bearing capacity is application of a semi-empirical equation proposed for this purpose by Terzaghi & Peck.(2) This equation may be written in the following form.

$$p_{ult} = \overbrace{C_1 N_c c}^{\text{cohesion}} + \overbrace{C_2 \gamma_1 N_\gamma B}^{\text{solid friction}} + \overbrace{\gamma_2 N_q D_f}^{\text{surcharge}} \tag{38}$$

In this equation the first quantity in the right hand member represents the contribution to bearing value of shearing strength due to cohesion, the second, strength due to solid friction and the third, the increase in strength due to surcharge loading. The terms C_1 and C_2 are constants dependent upon type of footing. The N terms are factors or coefficients which are represented as being dependent only on values of ϕ, the angle of internal friction. For the condition $\phi = 0$, these factors have values respectively of 5.77, 0 and 1.0. Other N values are obtainable from diagrams provided for such purpose. Unit cohesion is represented in the equation by the term c, footing size or breadth by B, γ_1 and γ_2 are respectively unit weight of soil below and above footing grade and D_f is depth of surcharge loading.

Equation (38) implies that (except for surcharge effect) the ultimate bearing capacity of a cohesive soil (for which ϕ and hence $N_\gamma = 0$) is dependent only on unit cohesion c, and is independent of footing width B, while for cohesionless soil it is indicated that ultimate capacity is a direct function of footing size. Surcharge effect for both soil types is represented as increasing the ultimate capacity in direct proportion to depth of structural backfilling D_f, the effect being considerably greater however for granular soils than clays because of the difference in the values of N_q which would be used in these two cases.

If contact pressures are to be restricted so as to eliminate danger of reaching the ultimate bearing capacity, a safety factor, SF is introduced, to give the expression,

$$p'_{all} = \frac{p_{ult}}{SF} \tag{39}$$

Safety factors which have been recommended for use in this connection range from 2 to 3, a value as low as 1.5 being occasionally considered justifiable. The contact pressure given by equation (39) is "allowable" in-so-far as security against rupture is concerned but it may or may not be acceptable with respect to settlement. It is for this reason that it has been designated p'_{all} rather than p_{all}.

The somewhat conflicting results obtained from considerations of soil compression and rupture theory respectively may be compared by reference to illustrations[3] such as Fig. 8. The curves in Fig. 8(a) relate to clay soils, those in Fig. 8(b) to granular soils. It will be noted that if rupture theory alone is considered, greater pressures (at least in cohesionless soils) would be allowed on large footings than on small ones, whereas for equal settlement the opposite design principle should be followed. Not shown in the figure is a similar disagreement with respect to surcharge, rupture theory as expressed in equation (38) indicating that contact pressures may increase with surcharge, compressibility theory indicating the reverse.

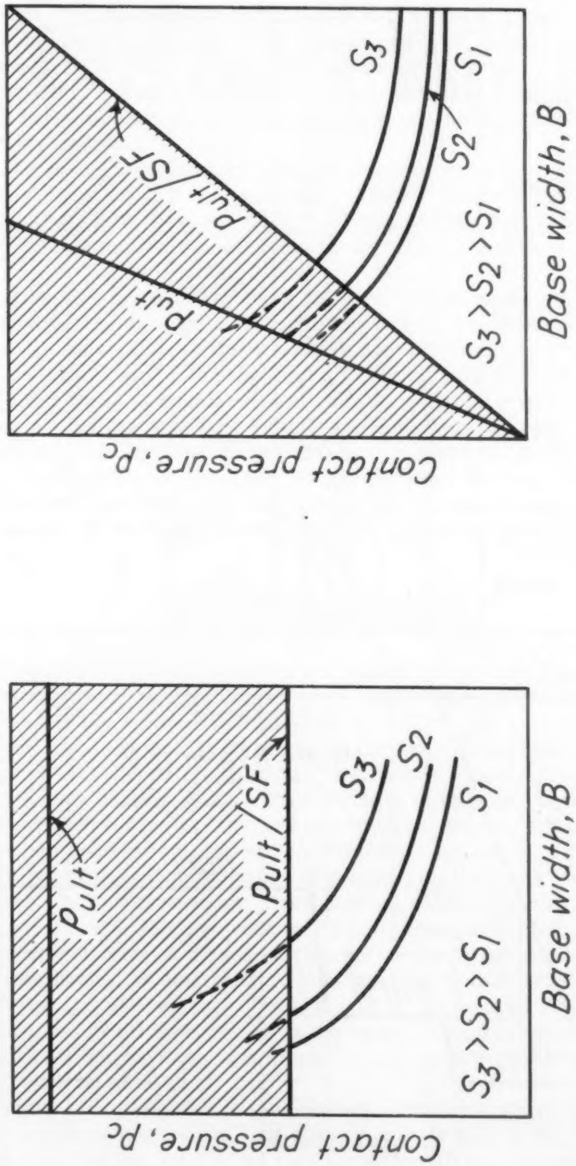
Fig. 8 helps to illustrate another point, namely that in-so-far as limitation of contact pressure to provide security against soil rupture is required at all, it is needed only in determining the size of the smallest of a group of footings. All other footings should then be proportioned for equal settlement due to one-dimensional soil compression, not shear. It will be noted that this conclusion is valid for both granular and cohesive soil types.

While the importance of considering soil rupture is not reduced thereby, the above finding increases the difficulty of justifying elaborate or expensive procedures for ultimate bearing value determination since this is of interest only in the design of one footing. The practical difficulties in applying equation (38) include the following. It is necessary among other things, to have data on unit cohesion and peak friction angle of the soil in its natural condition. It is not customary in routine site investigations to obtain the undisturbed soil samples and to conduct the laboratory tests necessary to provide such data. In cohesionless soils it would be very difficult in any case to carry out sampling and testing operations for peak strength determination since peak strength is a function of in-place density and thus requires undisturbed samples. A further consideration is that even if ultimate bearing capacity is determined, the need remains for obtaining data on soil compressibility and settlement. It would, therefore, be advantageous, if possible, to check on the possibility of soil rupture by some relatively simple method and then proceed directly to the problem of proportioning footings for equal settlement.

Presumptive Bearing Values

A requirement in any foundation design regardless of theoretical considerations, is compliance with applicable provisions of a building code. Codes establish what are known as "maximum allowable presumptive bearing values" for each class of soil. Such values would, therefore, establish a "ceiling" on the contact pressures which may be used for any footing. A comparison of presumptive bearing values from the Buffalo, New York building code and ultimate bearing values computed by equation (38), is given in Fig. 9. For cohesionless materials, the ultimate bearing value is represented to be a function of footing size whereas presumptive values are not. For this and other reasons, a presentation of data in tabular form is convenient for these materials, whereas a graphic form is effective with clays.

The ultimate bearing values for the granular soils were computed on the basis of assumed friction angle values of 34°, 38°, 42°, and 46° respectively for the loose, firm, compact, and very compact categories. In each case, a surcharge equivalent to two feet of material with a unit weight of 120 pounds per cubic foot, was assumed as the absolute minimum to be expected in practice. N_γ values for loose or soft material were used for the loose category, values for dense of stiff soils used for the remainder.



Cohesive soil
(a)

Cohesionless soil
(b)

FIG. 8

	Br'g Values, Cohesionless Soil - T/ϕ				
	Ultimate		Presumptive		
	B-ft.	P_{ult}	Fine Sd.	C-m. Sd.	Sd. & Grv.
Loose (0 to 10 blows)	2 4 6 8	1.6 2.1 2.5 2.9	$1\frac{1}{2}$	3	4
Firm (11 to 30 blows)	2 4 6 8	5.5 9.8 14.2 18.5	2	4	5
Compact (31 to 50 blows)	2 4 6 8	8.4 15.6 22.8 30.0	3	5	6
Very Compact (Over 50 blows)	2 4 6 8	22.2 43.2 64.2 85.2	-	6	10

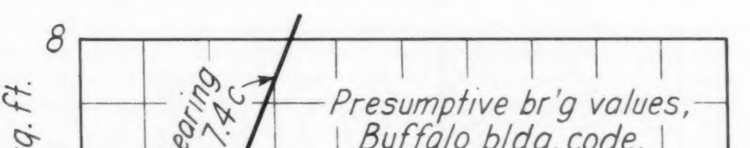


FIG. 9

Presumptive bearing values for granular soils are all less than the ultimate bearing values except for loose, coarse or medium sand and loose, sand and gravel. Since presumptive values of this order of magnitude for these two apparent exceptions are regularly used in practice, it may be argued that the ultimate bearing values are overconservative. With these two possible exceptions, however, it would appear that the presumptive bearing values for granular soils in general, are less and in most cases, substantially less than the ultimate values.

For clay soils, ultimate bearing values were computed on the basis of the correlation between unit cohesion and standard penetration resistance values given in Table 22 of "Soil Mechanics in Engineering Practice" by Terzaghi & Peck. All presumptive values given by the Buffalo code are less than these values.

On the basis of this evidence, it appears that security against soil rupture under the smaller footings in an ordinary design can be obtained simply by adopting the code value as the allowable contact pressure for the footing with the smallest total load. Footing sizes for other columns can then be determined on the basis of equalizing settlement, with the result that contact pressures for all but the smallest footing would be less than the code value. It is suggested that under these circumstances, somewhat liberal rather than extremely conservative allowances for presumptive bearing values may often be justified and that the sampling, testing and engineering analysis required for evaluation of ultimate bearing capacity are often unnecessary.

Recommended Procedure for Determining Soil Bearing Values

An apparently direct procedure for bearing value determination is utilization of charts such as Fig. 5 from which allowable pressure index values may be obtained in the manner indicated on the chart. This would require evaluation of soil compressibility in terms of the bearing capacity index $\underline{C}$, then selection of an appropriate tolerable settlement value $\underline{S}$, and thereafter, $\underline{K}$ values for each footing width. The difficulty of this approach as previously stated, is that the results obtained are influenced to an excessive degree by assumptions made as to tolerable settlement. It will often be the case too that only very approximate values of the bearing capacity index are readily obtainable. In this connection, the following comments may be made.

The values of $\underline{S}$ which appear in the equations presented above, represent total, gross settlement. Total settlement in itself, providing it is kept within reasonable limits, is generally of less interest to the structural engineer than differential settlement. Thus, if contact pressures are limited so that the settlement of all foundations is the same, it makes very little difference in most cases whether the settlement is half an inch, three quarters of an inch, or even an inch.

A further consideration is that contact pressures must be limited not only with reference to settlement, but also to comply with code provisions. If presumptive bearing values, as recommended above, are to be used in any case, as the basis for proportioning the smallest of a group of footings, it may be advisable to make this the basis for selecting a tolerable settlement value as well. In such a case, the following procedure would be adopted.

As an initial step, the size of the smallest footing would be determined by dividing the smallest column load by the maximum allowable presumptive bearing value.* The footing size obtained in this way is represented by point a in Fig. 10(a). The next operation would be to determine the allowable pressure index for this footing by substituting the maximum allowable presumptive bearing value for the term p'_c in equation (37). With the K and B values obtained in this manner and with a C value from Fig. 3 or from tests, the indicated gross settlement for the smallest footing can be determined from diagrams like Fig. 5. The larger footings would then be designated by reference to curve A, Fig. 10(a), constructed on the basis of K values for the same settlement. However, if the indicated settlement appears to be excessive, a curve for a lesser amount of settlement such as curve B must be constructed and a smaller contact pressure used in designing the smallest footing. The contact pressure and size of the smallest footing would then be as indicated by point b in Fig. 10(a).

For convenience in proportioning footings, it is advantageous to prepare diagrams like Fig. 10(b) showing the relation between footing width and total column load rather than contact pressure. Such a diagram also may be used as the basis for certain comparisons. For example, if all footings were proportioned for equal contact pressures according to the prevailing custom, the relationship between column load P and footing width would be as represented by curve B in Fig. 10(b). The equation for such a curve is,

$$B = B_{min}\sqrt{\frac{P}{P_{min}}} \tag{40}$$

As an alternative, it might perhaps be suggested that footing widths should vary directly with total column load. Curve C would represent this condition. The equation for curve C is

$$B = B_{min}\frac{P}{P_{min}} \tag{41}$$

With footings designed for equal settlement, the intermediate curve A is obtained. Although this matter has not yet been completely explored, it appears that for typical conditions, diagram A is a straight line, whereas curve B, representing more or less standard practice, is parabolic. When the relation between column load and footing width plots as a straight line, it is evidently necessary to determine only two footing sizes. The size of the smallest footing which is required in any case, would be determined by conventional methods according to present practice. The size of the largest footing could then be determined by the procedures recommended above and all intermediate footing sizes by straight line interpolation. While the actual design procedure would be more complicated due to the need for considering different depth and surcharge conditions, the basic principle can, as noted, be reduced to very simple terms.

*If and when desired, this same procedure could be followed utilizing a limiting contact pressure obtained from considerations of soil rupture in place of a presumptive bearing value.

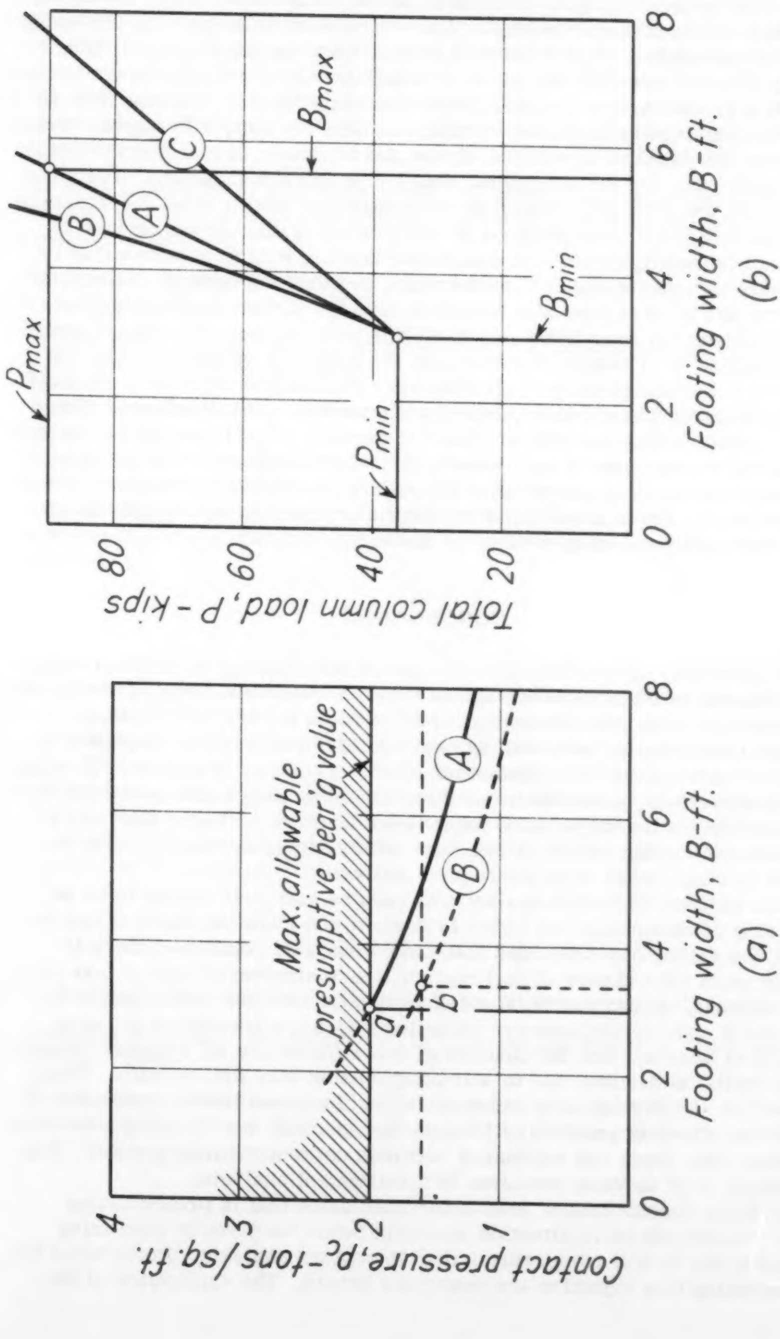

Total column load, P - kips
80
60
40
20
0
2
4
6
8
Footing width, B-ft.
(b)
Pmax
Pmin
Bmax
Bmin
A
B
C
Contact pressure, pc - tons/sq. ft.
4
3
2
1
0
2
4
6
8
Footing width, B-ft.
(a)
Max. allowable presumptive bear'g value
a
b
A
B

FIG. 10

One further consideration which is certain to be encountered in practice, but has not been specifically mentioned above, is the need, if any for special procedure for continuous footings. The entire discussion given in this paper relates to individual column footings, specifically square footings. When a loading of given intensity is applied to a continuous footing, the stress increment at a given elevation beneath is greater than would be the case with an individual footing due to stress overlap. On this account, with a given contact pressure, the depth of significant stress and thickness of compressible material is greater for the continuous than for an individual footing. For equal settlement, the contact pressure on the continuous footing must therefore, be less. An additional consideration in many cases is that the percentage of dead load is usually greater on continuous bearing wall foundations than on individual column footings. Thus for equal residual settlement, the contact pressure due to dead load only should be less for continuous than for individual footings. A complete analysis of this problem has not been attempted in this paper. It is simply reported that the available evidence points consistently to the desirability of allowing less contact pressure for continuous footings than for individual footings if settlement is to be equalized. There is also evidence that the ultimate bearing capacity of soil beneath a continuous footing is less than of soil beneath an individual footing. On the basis of experience alone, it appears that a 25% reduction of contact pressure values obtained by the above procedures for individual footings will usually be appropriate in proportioning continuous footings.

CONCLUSIONS

It is generally agreed that the objective of soil bearing value determination is elimination of the possibility of soil rupture, reduction of gross settlement to a tolerable value and elimination of differential settlement. Current analytical methods for establishing bearing value place major emphasis on the first criterion, namely elimination of the possibility of rupture. Development of convenient, practical procedures for estimating settlement especially in unstratified soils has to some extent been neglected. There has been a tacit assumption that with a satisfactory safety factor against rupture, settlement in many cases does not require analysis.

While analysis to insure against soil rupture may well appear to be an overriding consideration and hence to deserve the attention which it has received, the author has concluded that under practical conditions there is actually much less chance of soil rupture due to structural loading than has been imagined. In particular, if a reasonably careful site investigation is made and if code regulations are complied with as is mandatory in many cases, it is believed that the chances of soil rupture are nil whereas chances of differential settlement due to soil compression may well remain. Thus, the need for the development and application of rupture theory diminishes in importance whereas analysis of settlement especially for differing conditions of footing size, depth and surcharge becomes correspondingly greater. Rupture theory is of no value whatever in a settlement analysis.

The above considerations lead to the conclusion that in proportioning spread foundations more attention should in future be given to equalizing settlement due to soil compression than to rupture analysis. Procedures for accomplishing this objective are described herein. The application of the

recommended methods in typical situations has lead to certain conclusions about limitation of contact pressure in proportioning structural foundations. When settlement is due chiefly to one dimensional soil compression, it appears that the allowable contact pressure is a function of unit soil weight and a term designated the allowable pressure index. The value of this index depends on footing breadth, depth and surcharge, on soil compressibility and on the tolerable settlement. Using this principle it can be shown among other things, that ideally,

a) With ground water at footing grade, the contact pressure should be no more than half the value allowed when ground water is below the depth of significant stress.

b) Contact pressure should vary inversely with footing size and directly with footing depth.

c) The total column load on surcharged footings should be less than that on similar footings without surcharge.

It may be noted that the above conclusions are independent of soil type and vary only in degree.

It is further concluded that there is greater need of evaluating soil compressibility in a site investigation than shearing strength. For purposes of characterizing soils with reference to compressibility a useful parameter appears to be the bearing capacity index, C, a dimensionless term which is defined as

$$C = \frac{1+e}{C_c}$$

At the present time there appears to be a reasonable prospect of correlating this index with textural classification and in-place density, soil characteristics which are or should be established in any case even in a routine investigation. This prospect is one reason for entertaining the hope that procedures described herein which have at least some rational basis, may eventually be reduced to such practical and convenient form that they can be utilized in routine work. At present, only empirical methods can be used in the great majority of site investigations.

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3. These illustrations are in the form originated by D. W. Taylor "Fundamentals of Soil Mechanics," John Wiley & Sons, 1948.

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SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

INVESTIGATION OF UNDERSEEPAGE—MISSISSIPPI RIVER LEVEES

W. J. Turnbull,[1] F. ASCE and C. I. Mansur,[2] F. ASCE

This is one of three companion papers on underseepage and is referred to as companion paper No. 1. The second paper is entitled "Design of Underseepage Control Measures for Dams and Levees" and is referred to as companion paper No. 2. The third paper is entitled "Construction and Maintenance of Underseepage Control Measures" and is referred to as companion paper No. 3.

SYNOPSIS

Seepage and sand boils landward of Mississippi River levees have been a problem during major high waters. After the 1937 high water, the Mississippi River Commission initiated a general study of underseepage and various methods of its control. Its specific purposes were to develop a better understanding of the phenomena of seepage beneath levees and of factors influencing underseepage, to obtain information pertinent to analyses of underseepage, to develop and evaluate control methods, and to develop formulas and criteria for their design.

The studies reported herein include a compilation of past underseepage reports; exploration and geological studies of numerous sites where underseepage was a serious problem in 1937; installation of piezometers at selected sites to measure substratum pressures; field pumping tests to determine the permeability of the sand aquifer; theoretical, model, and prototype studies of relief wells, partial cutoffs, and landside berms for controlling underseepage; and observation and measurement of natural seepage during the 1950 high water.

Note: Discussion open until January 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2136 is part of the copyrighted Journal, of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 4, August, 1959.

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From the theoretical, model, and prototype studies it was concluded that:

a. Sand boils and subsurface piping along the Mississippi River levees are the result of excess hydrostatic pressure and seepage through deep pervious strata underlying the levees. The severity of underseepage, both excess hydrostatic pressure and seepage flow, is dependent upon the head on the levee, source of seepage, perviousness of substratum, and characteristics of the landside top stratum.
b. There is a definite correlation between surface geology and the location and occurrence of underseepage and sand boils.
c. Seepage flow and hydrostatic heads landward of a levee can be estimated from seepage formulas, and/or piezometric data, and a knowledge of riverward and landward foundation characteristics.
d. Removal of the natural top blanket riverward by borrow operations has aggravated the underseepage problem along Mississippi River levees. Except where clay several feet thick was left in place, the source of seepage was in the riverside borrow pits.
e. Underseepage can be controlled by properly designed and constructed landside seepage berms, relief wells, and riverside blankets.

The design and construction of underseepage control measures for dams and levees are considered in companion papers Nos. 2 and 3, respectively.

The Underseepage Problem and Investigations Made

Excessive hydrostatic pressure and seepage, in the form of sand boils that actively pipe material, landward of levees, always pose a threat to the safety of a levee during high water, and have been known to cause some levee crevasses.(10) Although no crevasses of Lower Mississippi River levees have been positively attributed to sand boils or piping since 1913, a failure at Weecama, La., in 1922 is believed to have been the result of underground piping, and subsurface piping almost caused a levee crevasse at Greenville, Miss., in 1929. Excessive seepage and sand boils also occurred along numerous reaches of the Lower Mississippi River levees during the flood of 1937. Subsequently many of these levees were enlarged and landside seepage berms and/or sublevees were constructed at known critical seepage areas. However, when these berms and sublevees were designed (1937-1940) little information was available regarding the characteristics of the foundation soils, the relation between geological features and underseepage, or rational methods for analyzing subsurface seepage. Because of this lack, the Mississippi River Commission, in September 1940, initiated a general study of underseepage and its control along Lower Mississippi River levees.

Purpose of Study

The basic purposes of the investigation were (a) to develop a better understanding of the phenomena of seepage beneath levees and of the factors that influence underseepage, (b) to obtain information pertinent to analyses of underseepage, and (c) to develop and evaluate methods of underseepage control, and to develop formulas and criteria for their design.

Scope of General Study and Investigations Made

Since initiation of the study, numerous investigations relating to the problem of underseepage and its control have been made. These studies have included a review and compilation of all underseepage reports made during and since the 1937 high water;[10] exploration and geological studies of numerous sites where underseepage was a serious problem in 1937; installation of piezometers at selected sites to measure substratum pressures beneath and landward of levees; field pumping tests to determine the permeability of the sand aquifer at certain sites;[7,13] theoretical, electrical-analogy, sand model, and prototype studies of relief wells,[9,12] partial cutoffs,[11] and landside berms for the control of underseepage; and observation and measurement of natural seepage at certain locations during the 1950 high water.

The first phase of the study (1941) consisted of an investigation of seven sites in the Memphis District, CE, where excessive underseepage and sand boils had occurred during the 1937 high water. The results of this study showed that the sand boils at these sites could be attributed primarily to excessive artesian pressures landward of the levee for the thickness of the existing landside top stratum and the existence of a pervious substratum of sand 75 to 150 ft thick which offered relatively free passage beneath and landward of the levee for hydrostatic pressure and seepage from the river and riverside borrow pits.

Geological Investigations.—The results of the 1941 study indicated the need for more geological information for a proper understanding of factors affecting the occurrence of underseepage. Accordingly, Dr. H. N. Fisk was retained in 1941 by the MRC to make a geological study,[2,3] at several sites in the Memphis, Vicksburg, and New Orleans Districts where underseepage had been a major problem during the 1937 high water.

Piezometer Sites.—The study of seven sites in the Memphis District also indicated the need for more specific information and data regarding the development of substratum hydrostatic pressures, the distance from the levee to the effective source of seepage entry, and the relation of these factors to underseepage and sand boils. In order to obtain this information, systems of piezometers were installed in 1942-1948 at sites along the Mississippi and Red Rivers shown in Fig. 1. Fairly complete piezometer readings were obtained at all of the sites during a high water in 1950.[16]

The natural seepage emerging landward of the levees was also measured at Gammon, Commerce, Trotters 51, Trotters 54, Stovall, and Baton Rouge sites during the 1950 high water.

Review of Underseepage and Crevasse Data.—As part of the general underseepage study, a compilation[10] was made of all known crevasse and underseepage data from the records of the Mississippi River Commission and the Memphis, Vicksburg, and New Orleans Districts. Field investigations at the locations of levee crevasses known to have been caused by underseepage would have been desirable. However, the conditions that resulted in failure of the levees were destroyed by the scour caused by the crevasse. In view of the elapsed time, lack of precise records, and difficulties involved in quantitative analyses, no investigations were made at old crevasses.

Sand Model Studies of Relief Wells.—A number of sand models were also constructed to study the phenomenon of underseepage and its control by means of relief wells.[9,12] The purpose of these model studies were to investigate the operation of relief wells, and to measure well and seepage flows, and

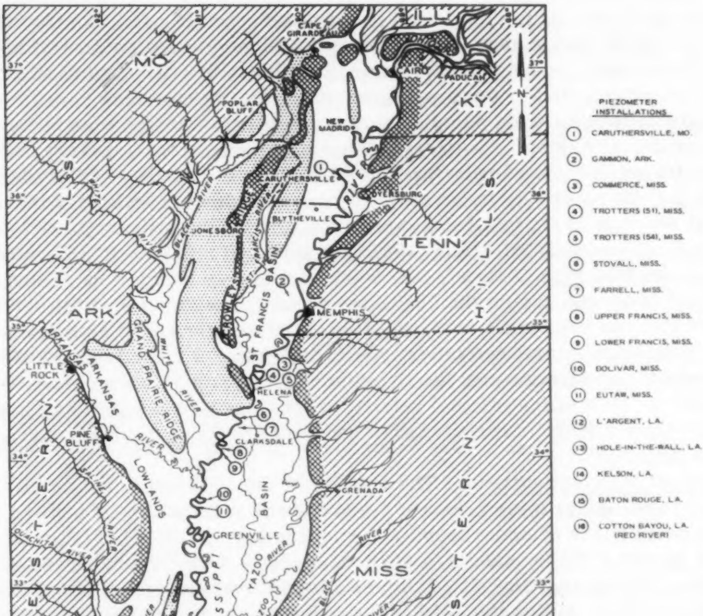


Fig. 1. Plan of alluvial valley of the Mississippi River and location of piezometer installations

landward substratum hydrostatic pressures with and without relief wells in operation, for various foundations, seepage entrances, and landward top strata representative of conditions commonly encountered in the Lower Mississippi River Valley.

In the models, relief wells with proper spacing and penetration effectively reduced excess hydrostatic pressure landward of levees underlain by a pervious foundation for a wide range of foundation conditions. With adequate well spacing and penetration, uncontrolled seepage normally emerging landward of a levee (without wells) was materially reduced, although the total underseepage flow was increased by about 20 to 40 per cent for a model typical of conditions along Lower Mississippi River levees.

Sand and Electrical-Analogy Model Studies of Partial Cutoffs.—A study[11] was also made of the effect of partial cutoffs, installed along the riverside toe of a levee, on underseepage and hydrostatic pressures beneath and landward of a levee. Various foundation and seepage entrance conditions were selected for study as representing, at least qualitatively, certain limiting conditions commonly encountered in the Lower Mississippi River Valley. The methods of analysis used included sand and electrical models, graphical analyses, and mathematical computations. These studies showed that partial cutoffs had relatively little effect on the reduction of underseepage or substratum hydrostatic pressures for the conditions tested.

Field Installation of Relief Wells.—In the summer of 1950 relief wells were installed at Trotters 54, Miss., for the purpose of making a full-scale field test of the efficacy of a relief well system for controlling underseepage. This well system operated very satisfactorily during the high water in 1951 and 1952.[13] Piezometer readings and seepage observations made during both of these high waters indicated that the well system reduced substratum hydrostatic pressures landward of the levee to a small fraction of the head of the levee, and also intercepted a large portion of natural seepage which otherwise would have emerged landward of the levee.

Field Installation of Partial Cutoff.—Another phase of the investigation consisted of the development of a machine and procedure for installing an impervious partial cutoff 30 ft deep or more along the riverside toe of a levee. This project was carried out by the Memphis District, CE, during 1946-1951 and culminated in the completion of a 40-ft cutoff along a 1400-ft reach of levee at Trotters 51, Miss., after which the project was discontinued. No performance data are available for the cutoff installed.

Seepage Berms.—The most commonly used method for safeguarding levees along the Lower Mississippi River against the hazards of sand boils and subsurface erosion is the construction of seepage berms along the landside toe of the levees. Most of these berms were designed and constructed in 1937-1940. No data are available regarding the substratum pressures that existed prior to construction of seepage berms to compare with pressures measured by piezometers installed after the berms were constructed.

Field Pumping Tests.—Field pumping tests were conducted at Commerce and Trotters 54 for the purpose of determining the overall permeability of the pervious substratum.

During installation of the well system at Trotters 54 in 1950, pumping tests were performed to determine the flow for various drawdowns in the well, head loss through the filter and well screen, and the permeability of the pervious sand stratum.[13] Since then comprehensive pumping tests have been made on wells installed along the levees in the St. Louis District, CE,[15] and at the

site for a lock and a control structure to be built in conjunction with the control of Old River south of Natchez, Mississippi.

Geology of the Lower Mississippi River Valley and Its Influence on Underseepage

Geological studies of several sites along the levees where underseepage had been a problem during previous high waters showed a definite correlation between the distribution of alluvial deposits of sand, silt, and clay with the location and occurrence of underseepage and sand boils. Thus, in order to properly understand and analyze the underseepage problem, it is necessary to have an understanding of the alluvial fill in the Mississippi River Valley.

The Alluvial Valley of the Lower Mississippi River

The alluvial valley of the Lower Mississippi River begins near the confluence of the Mississippi and Ohio Rivers, Cairo, Ill., and extends to the Gulf of Mexico (Fig. 1).

The alluvial deposits in the Lower Mississippi River Valley fill a trench ranging in depth from about 100 ft in the upper part of the valley to 400 ft near the Gulf. The origin of this buried canyon is attributed to changes in sea level caused by the glaciers which were formed during the Pleistocene epoch. It is estimated that when the glaciers reached their maximum extent sufficient water was withdrawn from the ocean to lower sea level by about 400 ft. A somewhat idealized illustration of the entrenched valley and alluvial fill is presented in Fig. 2.

The Alluvial Fill

As the glaciers of the late Wisconsin stage began to melt, some 30,000 years ago, the sea level gradually rose to its present position causing the entrenched valley to become filled with a series of sandy gravels, sands, silts, and clays that can be grouped into two broad units: (a) a sand and gravel substratum, and (b) a fine-grained top stratum. The alluvial materials are generally underlain by relatively impervious Tertiary deposits.

The Pervious Substratum.—During the gradual rise of sea level accompanying final retreat of glacial ice, the Mississippi River became an overloaded braided stream in which large quantities of gravel-bearing sands were deposited. As sea level continued to rise and the deposits on the floor of the entrenched valley continued to thicken, stream slopes were progressively reduced and both the quantity and grain size of the materials transported by the river decreased. The gravel-bearing sands were succeeded by coarse sands grading upward into progressively finer materials and terminating in deposits of very fine sand. The substratum sands are quite pervious and have a high seepage-carrying capacity.

The sandy alluvium generally ranges in thickness from 75 to 150 ft. North of the Louisiana-Arkansas boundary, medium sands are within 5 to 20 ft of the ground surface, but south of this line the thickness of the overlying materials increases and in the vicinity of Houma, La., "clean" sands are from 50 to 100 ft below the surface.

The Top Stratum.—Some 6000 years ago, sea level reached essentially its present position, and the sedimentary load carried by the river soon became

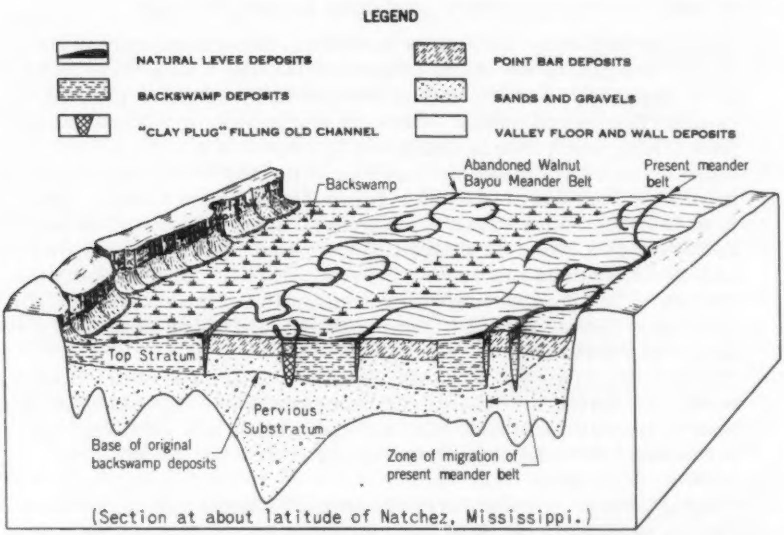


Fig. 2. Block diagram of alluvial valley of the Lower Mississippi River

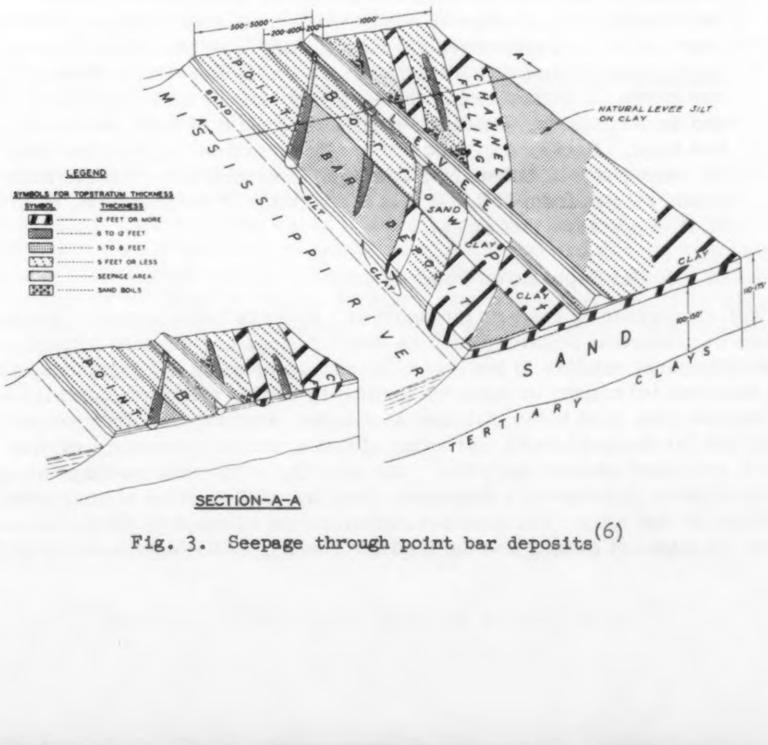


Fig. 3. Seepage through point bar deposits[6]

substantially adjusted to the slope and velocity. Rapid filling of the entrenched valley ceased, and the former braided channel was replaced by a meandering stream which deposited sediments consisting of point bar, channel fill, natural levee, and backswamp deposits, laid down in meander belts.

a. Point bar deposits. As a river meanders, deposition takes place on the inside bank where low sandy ridges are built up with intervening elongated depressions which usually become filled with fine-grained deposits. Ridges and swales formed in this manner are known as point bars. Long sandy ridges separated by clay- and silt-filled depressions more or less paralleling the margins of former river courses are also prominent features of shifting river valleys. These features appear to have been formed in migrating channels rather than on points and are called channel bars and slough fillings. The upper part of these ridge-and-swale deposits are usually covered with clay silts and silty clays which are laid down during gradual migration of the river channel from its former banks. Slough fillings normally are about 10 to 20 ft deep and 20 to 500 ft wide.

b. Channel-fill deposits. When the river abandons its former course as a result of a cutoff, the central and lower portions of the old cutoff channel usually become filled with silts and clays which are relatively impermeable. Such deposits in the Lower Mississippi River Valley may be 50 to 125 ft deep.

c. Natural levees. When the river overtops its banks the water spreads out, the velocity is checked, and deposition of a portion of the sedimentary load results. In this manner, long ridges known as natural levees are formed on the outside of meander loops and along both banks in straight reaches. Natural levee deposits found in the Lower Mississippi Valley range in composition from sandy silts to silty clays. They range in height above the surrounding floodplain from 5 to 10 ft.

d. Backswamp deposits. Low-lying areas on the landside of natural levees are known as backswamps. These areas receive only quiet flood waters and as a result the sediments deposited consist of fairly uniform silts and clays. Backswamp deposits create an almost impervious block to the emergence of subsurface seepage. The thickness of backswamp deposits ranges from 15 to 20 ft in the vicinity of Helena, Ark., to 20 to 70 ft in southern Louisiana.

Effect of Alluvial Deposits on Underseepage

The emergence of seepage landward of a levee is influenced by: (a) configuration of geological features, such as swale fillings and channel fillings and their alignment relative to the levee; (b) characteristics and thickness of the top stratum; (c) cracks or fissures formed by drying and other natural causes; (d) borrow pits, post holes, seismic shot holes, ditches, and other works of man; and (e) decay of roots, uprooting of trees, animal burrows, crayfish holes, and other organic agencies. The severity of the underseepage along a reach of levee is frequently dependent upon the configuration of geological features in that area. The greatest concentration of seepage always occurs along the edges of swales and the landside levee toe as illustrated in Fig. 3.

Occurrence and Analysis of Underseepage

Development of Underseepage and Sand Boils

Whenever a levee is subjected to a differential hydrostatic head of water as a result of river stages higher than the surrounding land, seepage enters the pervious substratum through the bed of the river, riverside borrow pits, and/or the riverside top stratum, and creates an artesian head and hydraulic gradient in the sand stratum under the levee. This gradient causes a flow of seepage beneath the levee and the development of excess pressures landward thereof (Fig. 4). If the hydrostatic pressure in the pervious substratum landward of the levee becomes greater than the submerged weight of the top stratum, the excess pressure will cause heaving of the top blanket, or will cause it to rupture at one or more weak spots with a resulting concentration of seepage flow in the form of sand boils (Fig. 5).

In nature, seepage usually concentrates along the landside toe of the levee, at thin or weak spots in the top stratum, and adjacent to clay-filled swales or channels. Where seepage is concentrated to the extent that turbulent flow is created, the flow will cause erosion in the top stratum and development of a channel down into the underlying silts and fine sands which frequently exist immediately beneath the top stratum. As the channel increases in size and/or length, a progressively greater concentration of seepage flows into it with a consequent greater tendency for erosion to progress beneath the levee. Although a number of levee crevasses have occurred as a result of sand boils, whether or not a specific levee will be crevassed as a result of critical substratum pressures and concentrated seepage in the form of sand boils or piping is practically impossible to predict.

The amount of underseepage and uplift hydrostatic pressure that may develop landward of a levee is related to the river stage, location of seepage entrance, thickness and perviousness of the substratum and of the landside top stratum, underground storage, and geological features. Other factors contributing to the activity of sand boils caused by seepage and hydrostatic pressure are the degree of seepage concentration and the velocity of flow emerging from the boils.

Underground storage has a significant effect on underseepage and excess hydrostatic pressures during relatively low high waters or high waters of short duration. If the ground water table is low at the onset of a high water, drainage into subsurface storage landward of the levee will reduce hydrostatic pressures and seepage rising to the surface. However, if the ground water table is high or the flood is of long duration, this factor will have little effect on substratum hydrostatic pressures. In general, piezometric data obtained

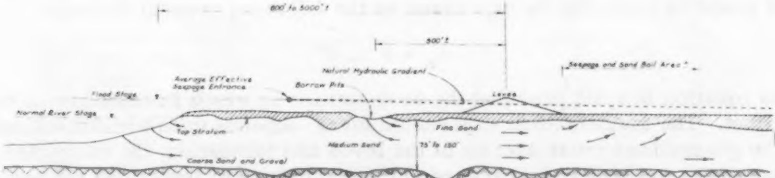


Fig. 4. Cross section of levee and alluvial valley(6)

Fig. 5. Sand boils in sack sublevee basin near Friars Point, Miss.,(6) 1937 high water

during the 1950 high water indicate that the ground-water storage landward of Lower Mississippi River levees will be filled by the time a high flood stage develops.

Computation of Seepage Flow and Substratum Pressures

Seepage flow and hydrostatic heads landward of a levee can be estimated from seepage formulas and/or piezometric data, and a knowledge of the top stratum characteristics both riverward and landward of the levee, and of the pervious substratum. However, the accuracy of results obtained from such formulas is dependent on the applicability of the formula to the condition being analyzed, the uniformity of soil conditions, and evaluation of the various factors involved in the computations.

Other factors that influence computation of seepage flow and substratum pressure, but do not lend themselves to theoretical analysis, are stratification of the foundation, lenticular deposits of silts and clays within the foundation, nonuniformity of the top stratum, and riverside or landside borrow pits.

For a levee underlain by a pervious foundation, the natural seepage Q_S per unit length of levee can be expressed by the following general formula

$$Q_s = \$ \, k_f \, H^* \tag{1}$$

This equation is valid provided the assumptions on which Darcy's law is based are met. The mathematical expression for $\$$ depends upon the dimensions of the generalized cross section of the levee and foundation, the characteristics of the top stratum riverward and landward of the levee, and the pervious

*See Appendix A for definition of notations and symbols.

substratum. Where the hydraulic grade line M beneath the levee is known from piezometer readings, seepage passing beneath the levee can also be determined from the formula

$$Q_s = M\, k_f\, d \tag{2}$$

The excess hydrostatic head h_o beneath the top stratum at the landside toe of the levee is related to the net head on the levee, the dimensions of the levee and foundation, permeability of the foundation, and the character of the top stratum riverward and landward of the levee. The head h_x beneath the top stratum at a distance x landward from the landside toe of the levee can be expressed in terms of h_o. When h_x is expressed in terms of h_o it depends only upon the type and thickness of the blanket and pervious foundation landward of the levee; the ratio h_x/h_o is independent of conditions riverward of the levee.

Expressions for $\oint$, h_o/H, and h_x/h_o for typical levee and foundation conditions along the Lower Mississippi River are presented in Fig. 6 and are discussed in the following paragraphs.

No Top Stratum.—Where a levee is founded directly on pervious foundation sands and no top stratum exists either riverward or landward of the levee (case 1, Fig. 6), $h_o = 0$. The severity of such a condition in nature is governed by the exit gradient and seepage velocity that develop at the landside toe of the levee; these can be estimated from a flow net.

Some idea of the safety of a levee against piping where no top stratum exists may also be obtained from comparison of a computed creep ratio and recommended empirical values thereof. The creep ratio can be computed by either of the following formulas proposed by Bligh[1] or Lane,[5] and the answers therefrom compared with the respective minimum values listed in Table 1.

$$\text{Bligh's creep ratio, } C = \frac{L_2}{H} \tag{3}$$

$$\text{Lane's "weighted" creep ratio, } C_w = \frac{1/3\ L_2 + \Sigma p}{H} \tag{4}$$

where Σp = sum of shortest vertical paths of seepage flow around cutoffs beneath a levee.

Table 1

Minimum Creep Ratios

Material	Creep Ratios: Bligh C	Creep Ratios: Lane C_w
Very fine sand or silt	18	8.5
Fine sand	15	7
Medium sand	--	6
Coarse sand	12	5
Fine gravel or sand and gravel	9	4
Coarse gravel including cobbles	4 to 6	3
Boulders with some cobbles and gravel	--	2.5

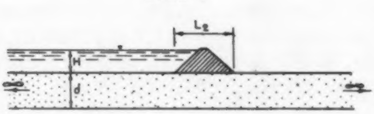
Basic Equations and Definitions

Seepage per unit length of levee .... $Q_s = f k_f H$

Head beneath top stratum at landside toe of levee .... h_o

Head beneath top stratum at x distance landward from landside toe of levee .... h_x

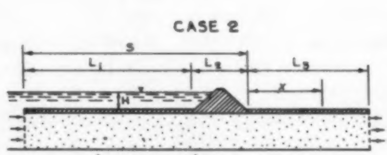
CASE 1



$$f = \frac{d}{L_2 + 0.86\,d}$$

$\frac{h_o}{H}$ and $h_x = 0$

CASE 2



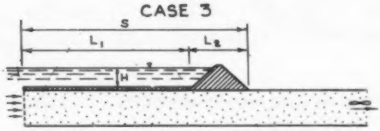
$$f = \frac{d}{L_1 + L_2 + L_3} = \frac{d}{s + L_3}$$

$$\frac{h_o}{H} = \frac{L_3}{s + L_3}$$

for $x \leq L_3$, $\frac{h_x}{h_o} = \frac{L_3 - x}{L_3}$

for $x \geq L_3$, $h_x = 0$

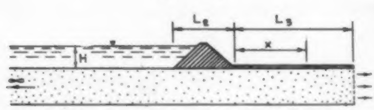
CASE 3



$$f = \frac{d}{L_1 + L_2 + 0.43d} = \frac{d}{s + 0.43d}$$

$\frac{h_o}{H}$ and $h_x = 0$

CASE 4



$$f = \frac{d}{0.43d + L_2 + L_3}$$

$$\frac{h_o}{H} = \frac{L_3}{0.43d + L_2 + L_3}$$

$$\frac{h_x}{h_o} = \frac{L_3 - x}{L_3}$$

Note:
Formulas for f and $\frac{h_o}{H}$ for cases 3 and 4 also are applicable where the indicated impervious top stratum is semi-pervious provided the effective lengths of blanket x_1 and x_3 are substituted in the formulas for L_1 and L_3 respectively.

FORMULAS FOR COMPUTATION OF SEEPAGE FLOW AND SUBSTRATUM PRESSURES - NO TOP STRATUM AND IMPERVIOUS TOP STRATUM

Fig. 6

Impervious Top Stratum.—A somewhat uncommon condition is that where the top stratum landward of the levee is almost completely impervious (cases 2, 3, 4, Fig. 6). Such a condition is approximated, however, where levees are founded on thick (>15 ft) deposits of clay, or silts with clay strata. For the condition of an impervious landside top stratum little or no seepage occurs through the top stratum and, if the top stratum is either infinite in landward extent or the pervious aquifer is blocked landward of the levee, no seepage occurs beneath the levee and $h_o = h_x = H$.

Conditions of no top stratum riverward or landward of a levee (cases 3 and 4, Fig. 6) are sometimes encountered where extensive borrow operations result in removal of all top stratum for a considerable distance from the levee.

Semipervious Top Stratum.—The condition most commonly encountered is that where a semipervious top stratum overlies the pervious substratum. The formulas given in Fig. 7 are based on the assumption that seepage flow through the top stratum is in a vertical direction and seepage in the pervious substratum is in a horizontal direction. These assumptions are essentially valid wherever the permeability ratio k_f/k_b exceeds 10. In nature this ratio usually exceeds 10; it ranged from about 100 to 5000 for the piezometer sites studied.

Values of h_o and h_x given by the equations on Fig. 7 are hydrostatic heads at the middle of the pervious substratum; where k_f/k_{bL} is less than 100 to 500, values of h_o and h_x immediately beneath the top stratum will be slightly less than those computed from x_3 because of the head loss resulting from upward seepage through the sand stratum.

In order to simplify the determination of h_x for various values of x, a graph of the relationship between h_x/h_o and x/x_3 has been plotted in Fig. 8 for $L_3 = \infty$, and for various values of x_3/L_3 for both a blocked and open exit at L_3. Once s, x_3, L_3, and the head h_o at the landside toe of the levee have been determined, the ratio h_x/h_o can be read from Fig. 8 for any particular x/x_3; then h_x can be computed from h_x/h_o.

Determination of Factors Involved in Seepage Analyses

Before any seepage analysis by means of theoretical formulas is possible, it is necessary to make certain simplifying assumptions and to generalize the foundation into a pervious sand stratum with a specific thickness and permeability and a semipervious top stratum with a uniform thickness and permeability. (However, the thickness and permeability of the top stratum may be different riverward and landward of the levee.) Seepage may enter the pervious stratum either at the river bank, through riverside borrow pits, and/or through the semipervious top stratum riverside of the levee. Seepage through the pervious substratum is assumed to be horizontal. Flow through the top stratum, or bottom of borrow pits, is assumed to be vertical. The levee, impervious or thick berms, and the portion of the top stratum immediately beneath them, are assumed to be impervious. In most of the theoretical formulas used in this paper it is assumed that the ground-water storage landward of the levee is essentially filled and that seepage through the top stratum and in the pervious sands is laminar. (However, seepage beneath a levee can be computed from certain formulas even though flow through the top stratum landward of the levee is no longer laminar.)

Some of the factors involved in seepage analyses may be determined or estimated by different methods, some more accurate than others. Methods of determining the necessary factors include the use of surveys, field

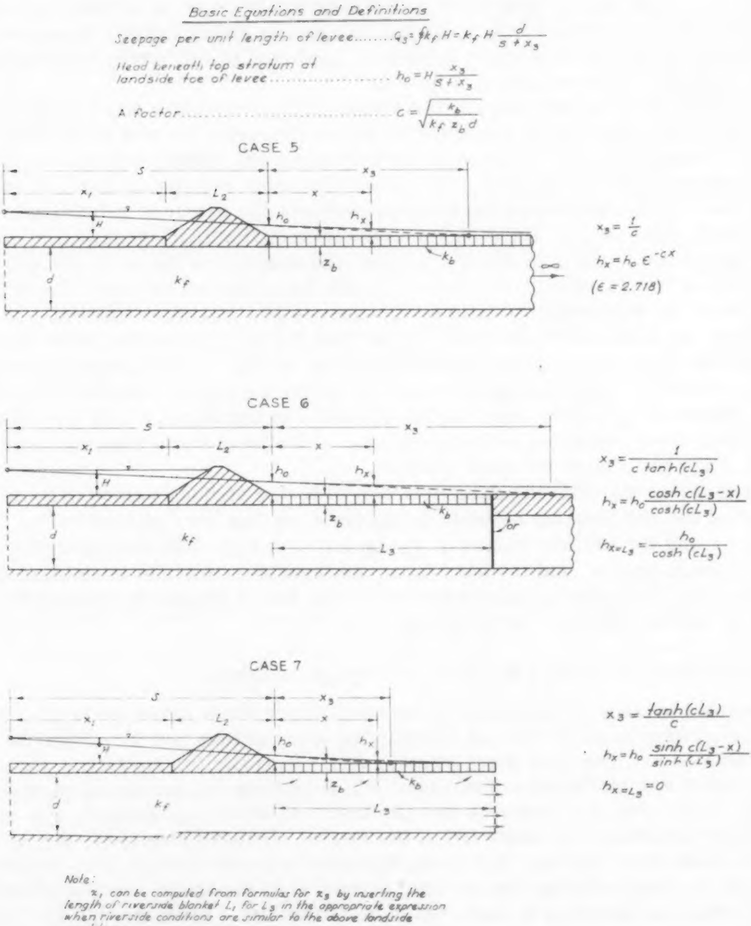


Fig. 7. Formulas for computation of seepage and substratum pressures for semipervious landside top stratum

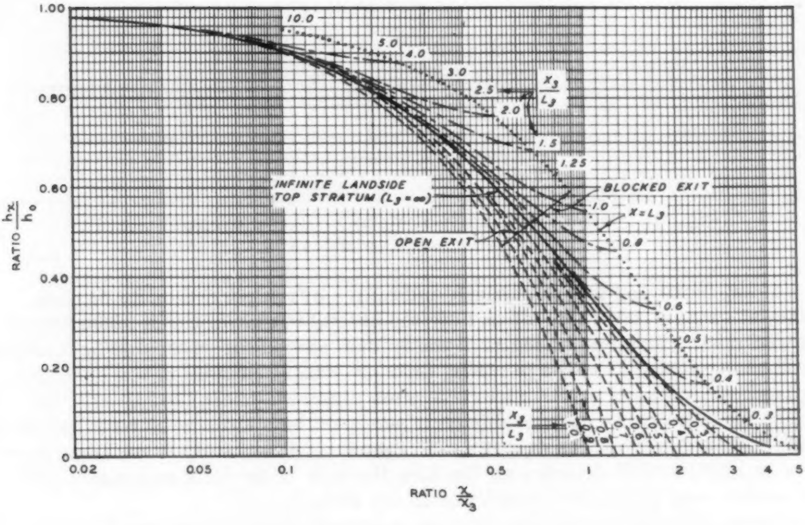


Fig. 8. Ratio between head landward of levee and head at landside toe of levee for levees founded on semipervious top stratum underlain by a pervious substratum

explorations, laboratory tests, field pumping tests, and piezometer systems. Methods of arriving at numerical values of these factors are discussed in the following paragraphs.

Net Head H.—The net head on a levee is the height of the flood stage above the tailwater or average low ground surface landward of the levee.

Length of Top Stratum Landward of Levee (L_3).—L_3 is usually considered to be infinite unless changes in geology or topography limit the emergence of seepage to a definite area. The distance to such a block created by high ground or a wide clay-filled slough can be ascertained from field reconnaissance, geological studies, aerial mosaics, borings, and/or topographic maps.

Hydraulic Grade Line (M).—M can best be determined from piezometers located beneath the levee where the seepage flow lines are essentially horizontal and the equipotential lines vertical.

Effective Thickness (z_b) and Permeability of Top Stratum (k_b).—The thickness of the top stratum both riverward and landward of a levee is usually determined by borings. Borings should be made to delineate the thickness and extent of any geological feature within 500 ft landward of the levee toe that may significantly affect the seepage analysis. The thickness of the top stratum in the bottom of landward ditches should also be determined.

The in-situ permeability of clay strata in the top blanket is related to the thickness of clay, whether or not it is at or near the surface or covered by natural levee deposits, and to a large extent to the presence of root holes, shrinkage cracks, minute fissures, and burrows of crayfish and small animals. Flow up through relatively thin (< 5 ft) clay strata near the surface is generally through these channels rather than through the pores of the soil. Tests on small samples of clay in the laboratory measure the permeability of the pores

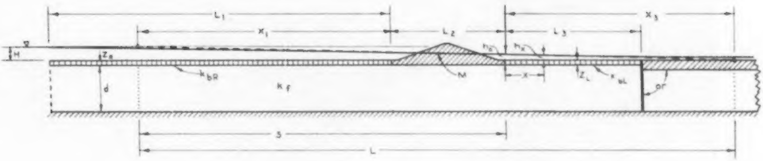


Fig. 9. Generalized cross section of levee foundation and symbols for seepage analysis

in the soil mass and are not usually indicative of the permeability of clay strata at or near the surface. A clay top stratum landward of a levee is considered more pervious during high water than one on the riverside because of the flushing action of seepage rising through small channels to the surface on the landside whereas on the riverside such small channels tend to silt up.

The in-situ vertical permeability of semipervious soils such as silty sand, sandy silt, and silt can be determined reasonably accurately from laboratory tests on undisturbed samples as the flow through these soils is usually laminar unless sand boils have developed in the area.

In seepage analyses the top stratum is usually generalized into a blanket of uniform vertical permeability with a specific effective thickness by transforming the actual thickness of various strata to another thickness with a certain permeability, as illustrated by the tabulation shown below. Transformation factors used in this study are given in Table 2.

Strata	Actual z_n	Actual $k_{v-n} \times 10^{-4}$ cm/sec	Thickness Transformation Factor $\frac{k_b}{k_{v-n}}$	Transformed, z_{b-n} for $k_b = 1 \times 10^{-4}$ cm/sec
Clay	5 ft	1	1.0	5.0
Sandy silt	8 ft	2	1/2	4.0
Silty sand	5 ft	10	1/10	0.5
Top stratum thickness z =	18 ft	--	----	Transformed $z_b = 9.5$

The effective vertical permeability k_{bL} of the top stratum landward of a levee can best be computed from observed hydrostatic heads beneath the landside top stratum, together with seepage measurements, and the following formula:

$$k_{bL} = \frac{Q_A \; z_{bL}}{h_x \; A} \; . \tag{5}$$

The permeability of the top blanket k_{bL} can also be computed from known characteristics of the pervious foundation and the effective seepage exit x_3 as determined from the hydraulic grade line in the pervious foundation beneath the levee using the following formulas:

Table 2

Thickness Transformation Factors for Top Strata

Soil Type: LMVD	Soil Type: Unified Soil Class. System	Transformation Factor
	Clay less than 5 ft in Thickness	
Clay	Fat clay (CH)	1
Silty clay	Lean clay (CL)	1
Clay silt	Silt (ML)	1
Sandy silt	Silt, sandy (ML)	3/4 to 1
Silty sand	Silty sand (SM)	1/5 if z < 10 ft; 0 if z >10 ft
Very fine sand	Fine sand	0
Alternating clay and silt strata with depth		1
	Clay more than 5 ft in Thickness	
Clay	Fat clay (CH)	1
Silty clay	Lean clay (CL)	1
Clay silt	Silt (ML)	1/2
Sandy silt	Silt, sandy (ML)	1/4 to 1/2 if z < 10 ft; 0 if z > 10 ft
Silty sand	Silty sand (SM)	1/10 if z < 10 ft; 0 if z > 10 ft
Very fine sand	Fine sand	0
Alternating clay and silt strata with depth		1

Where $L_3 = \infty$,

$$k_{bL} = \frac{z_{bL}\, k_f\, d}{x_3^2} \tag{6}$$

Where L_3 = a finite distance,

$$c \tanh cL_3 = \frac{1}{x_3} \tag{7}$$

where

$$c = \sqrt{\frac{k_{bL}}{k_f\, z_{bL}\, d}} \tag{7a}$$

In formulas 7 and 7a k_{bL} has to be determined by trial and error. (If $\frac{k_f}{k_{bL}}$ is less than 100 to 500, the value of k_{bL} computed from x_3 will be slightly low, because of the loss in head up through the aquifer at freely seeping sites.)

Where borrow pits, ditches, or channels exist within 200 or 300 ft of the landside levee toe, the thickness of top stratum used in computing seepage flows and substratum pressures should be based on the thickness of the top stratum adjacent to the ditch, unless the ditch or borrow pit is very wide. The allowable critical substratum pressure must be computed for both the thickness

of the top stratum at the toe of the levee and in the bottom of the ditch.

Effective Thickness d and Permeability k_f of Pervious Substratum.—The thickness of the pervious substratum (thickness of the principal seepage-carrying sand stratum below the top stratum and above the bottom of the entrenched valley) may be determined by means of deep borings or a combination of shallow borings and seismic or electrical resistivity surveys. (The thickness of very fine sand strata of low permeability that frequently exist between the top stratum and the principal seepage-carrying stratum is usually ignored in seepage and pressure computations.)

The average horizontal permeability k_f* of the pervious substratum is best determined by means of a field pumping test on a well that fully penetrates the pervious aquifer and using Eq. (8).

$$Q_w = \frac{2\pi\, k_f d\, (h_1 - h_2)}{2.30 \log_{10} \frac{r_2}{r_1}} \tag{8}$$

Where feasible, the flow in the well should be measured by means of a well flow meter(14) at major changes in sand strata. For soil conditions usually existing in the Lower Mississippi River Valley, the permeability of the pervious substratum is best computed from formula 8 for artesian flow. (The top stratum and upper fine sands are usually so much less pervious than the underlying deeper sands that they in effect create an upper impervious boundary. The lower impervious boundary is formed by the deep Tertiary materials.)

Where the permeabilities of different sand strata in the pervious substratum are significantly different (Fig. 10) the permeability of individual sand strata can be computed from the difference in the well flows in the screen at the boundaries of the sand stratum being tested using Eq. (8a).

$$k_{f(a)} = \frac{2.30\, q_a \log_{10} \frac{r_2}{r_1}}{2\pi\, d_a\, (h_1 - h_2)} \tag{8a}$$

The average k_f can be computed from the following formula:

$$k_f = \frac{d_a\, k_a + d_b\, k_b + d_c\, k_c + d_n\, k_n}{d_a + d_b + d_c + d_n} \tag{9}$$

The average horizontal permeability can also be determined from pumping tests on partially penetrating wells by using the straight line portion of the drawdown curve (with r plotted to a semilog scale) some distance from the well where flow lines to the well are essentially horizontal and are not affected by the curved pattern of flow in the vicinity of the well, and Eq. (8). If the pervious stratum is homogeneous and $k_V = k_H$, the average permeability can be approximated from Eq. (8), modified as follows:

$$k_f = \frac{2.30\, Q_w \log_{10} \frac{R}{r_w}}{2\pi\, d\, h\, G} \tag{8b}$$

*k_f in this paper is generally taken as k_H except in analyses of partially penetrating relief well systems or drainage trenches where a difference in k_f and k_H would have a significant bearing on the analysis.

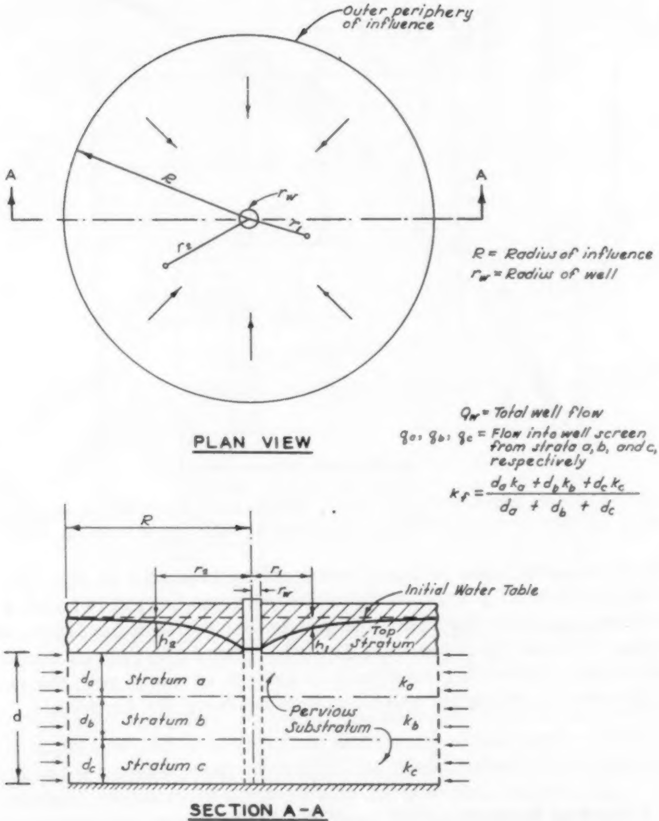


Fig. 10. Artesian flow to a test well

where G = ratio of flow of a partially penetrating to a fully penetrating well computed from Muskat's formula as determined from Fig. 11.

When it is not possible to determine k_f from pumping tests, it may be estimated from mechanical analyses or laboratory permeability tests on samples of sand taken by means of a Shelby tube or split spoon sampler in a "mudded" hole, or a piston-type bailer. If no gravel is present, nearly undisturbed samples of sand can be obtained with a Shelby tube sampler in a "mudded" hole. The next best sand samples are obtained with a piston-type bailer. Samples taken with either a Shelby tube or split spoon sampler in holes bored with drilling mud must be thoroughly washed of all drilling mud before testing.

Pumping tests have shown that the actual horizontal permeability of a sand stratum is 1.5 to 4 times greater than the permeability indicated by laboratory tests on remolded samples taken by any of the previously mentioned sampling methods.(7) An approximate empirical relationship between D_{10} and k_H

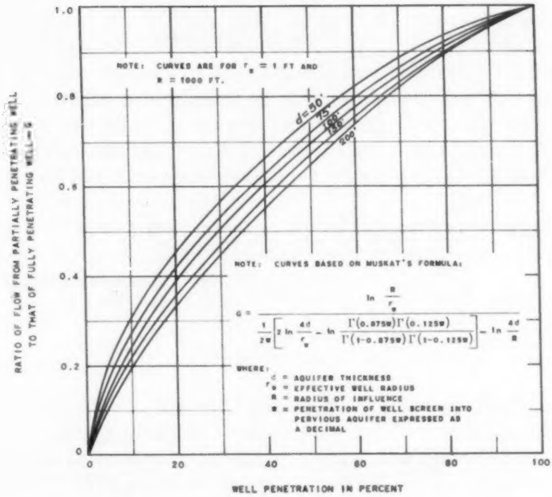


Fig. 11. Relation between flow from a partially penetrating artesian well in a homogeneous foundation and from a fully penetrating well

developed from pumping tests in the alluvial valley is shown in Fig. 12.

The average horizontal permeability of the pervious strata beneath a levee can also be estimated from the hydraulic grade line beneath the levee, total seepage passing beneath the levee, and Eq. (2). Where the rate of seepage flow per unit length of levee Q_A emerging in area A is measured, the permeability of the pervious substratum can be computed from the formula:

$$k_f = \frac{Q_A}{(M - M_A)\, d} \tag{2a}$$

Effective Source of Seepage Entry s.—The effective source of seepage entry into the pervious substratum, as illustrated in Fig. 9, is defined as that line riverward of the levee where a hypothetical open seepage entry face fully penetrating the pervious aquifer with an impervious blanket between this line and the levee would produce the same flow and hydrostatic pressure beneath and landward of a levee as will occur for the actual conditions riverward of the levee. It may also be defined as that line or point where the hydraulic grade line beneath the levee projected riverward with slope M intersects the river stage.

The best and most accurate method for determining s is to project graphically the hydraulic grade line M beneath the levee, as measured by piezometers installed in the pervious substratum beneath the levee, until it intersects the river stage producing the gradient. The value of s can also be determined from piezometric data using the following equations (see Fig. 13 for nomenclature):

$$s = \ell_1 + (H - h_1)\,\frac{(\ell_2 - \ell_1)}{(h_2 - h_1)} \tag{10}$$

$$s = \ell_1 + \frac{H - h_1}{M} \tag{10a}$$

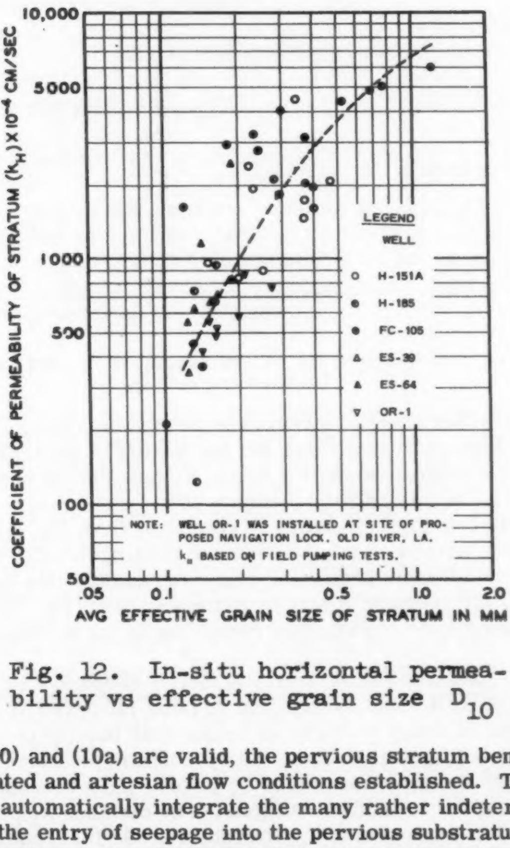


Fig. 12. In-situ horizontal permeability vs effective grain size D_{10}

Before Eqs. (10) and (10a) are valid, the pervious stratum beneath the levee must be saturated and artesian flow conditions established. These methods of determining s automatically integrate the many rather indeterminate factors that influence the entry of seepage into the pervious substratum, and were used for determining the source of seepage at the piezometer sites.

If the distance to the river $L_1 + L_2$ (Fig. 9) is known and there are no riverside borrow pits, s (which equals $x_1 + L_2$) can be estimated from the following formula:

$$x_1 = \frac{\tanh c\, L_1}{c} \tag{11}$$

where

$$c = \sqrt{\frac{k_{bR}}{k_f\, z_{bR}\, d}} \tag{11a}$$

Where a block, or wide, thick deposit of clay exists a certain distance riverward of a levee so as to prevent any entrance of seepage into the foundation beyond that point (Fig. 9), s (which equals $x_1 + L_2$) can be estimated from the following equation for x_1:

$$x_1 = \frac{1}{c \tanh cL_1} \tag{12}$$

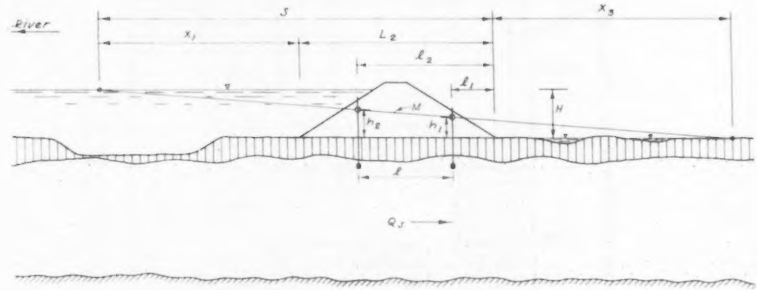


Fig. 13. Nomenclature for determination of s and x_3 from piezometer readings

Where two guide levees parallel a tributary stream or a floodway channel, and seepage into the foundation is divided and the bottom of the tributary stream or channel does not expose foundation sands, x_1 can also be computed from Eq. (12) wherein L_1 equals half the distance between the riverside toes of the levees. (The term c in Eq. (12) would be computed from Eq. (11a).

Where riverside borrow pits have been dug causing most of the impervious stratum over a considerable area to be removed, and the pits become the primary entrance for seepage, s may be estimated from Eq. (12) where z_{bR} and k_{bR} are values for the top stratum remaining in the bottom of the borrow pit.

Distance from Landside Levee Toe to Effective Seepage Exit x_3.—The effective seepage exit is defined as that line or point landward of the levee where a hypothetical open drainage face and an impervious blanket between this point and the levee toe would result in the same hydrostatic pressure at the levee toe and would cause the same amount of seepage to pass beneath the levee as would actually occur for existing conditions. The distance x_3 to this point is the intersection of an extension of the hydraulic grade line beneath the levee with the ground surface or tailwater (see Fig. 9). The best way to determine x_3 is by means of piezometers installed in the pervious substratum beneath a levee using the following expression (see Fig. 13 for nomenclature):

$$x_3 = h_1 \frac{(\ell_2 - \ell_1)}{(h_2 - h_1)} - \ell_1 \tag{13}$$

or

$$x_3 = \frac{h_1}{M} - \ell_1 \tag{13a}$$

As x_3 may vary with river stages as seepage develops, x_3 should be plotted vs river stage, and the estimated maximum value obtained from a curve of best fit should be used for seepage analyses.

The distance to the effective seepage exit can also be estimated from blanket formulas:

Where $L_3 = \infty$

$$x_3 = \frac{1}{c} = \sqrt{\frac{k_f\ z_{bL}\ d}{k_{bL}}} \tag{14}$$

where L_3 = finite distance to a block

$$x_3 = \frac{1}{c \tanh c L_3} \tag{15}$$

where L_3 = finite distance to an open exit

$$x_3 = \frac{\tanh c L_3}{c} \tag{16}$$

The relationship between c and the effective seepage exit length x_3 where the semipervious top stratum is infinite in landward extent (case 5, Fig. 7) has been computed from Eq. (14) and plotted in Fig. 14 for various values of k_f/k_b assuming d = 100 ft. The x_3 corresponding to values of d other than 100 ft can be computed from the equation

$$x_3 = 0.1 \sqrt{d}\; x_{3(d = 100)} \tag{17}$$

Where the landside top stratum has a finite length L_3 with either a block or open seepage exit at L_3 (see Fig. 7, cases 6 and 7, respectively), the effective exit length can be computed either from Eq. (15) or (16) or by multiplying x_3 ($L_3 = \infty$) by the factor shown in Fig. 15. Figs. 14 and 15 can also be used to evaluate the effective length of riverside blanket x_1 by using L_1, z_{bR}, and k_{bR} for L_3, z_{bL}, and k_{bL}, respectively.

Combinations of s and x_3.—Various combinations of s and x_3 for use in the design of underseepage control measures can be estimated from the reading of a single piezometer at a fairly high river stage, by determining the distances to the source of seepage entry and the effective seepage exit required to create an h_o equal to the observed head at the toe of the levee. The distance to the source of seepage can be estimated from reaches where piezometers have been installed perpendicular to the levee and where riverside soil conditions are of a similar nature.

Ratio k_f/k_{bL}.—The ratio k_f/k_{bL} can be computed from Eqs. (6) and (7) and values of x_3 determined from the hydraulic gradient beneath the levee without knowing either k_f or k_{bL}.

Critical Gradient i_c.—The critical gradient required to cause sand boils or heaving or flotation of the top stratum is usually defined as the ratio of the submerged unit weight of the soil comprising the top stratum and the unit weight of water. Homogeneous soils have the following approximate theoretical critical gradients:

Soil Type	i_c
Silty sand and silts	0.85
Silty clay and clay	0.80

The critical gradient required to cause sand boils in the field can best be determined by measuring the hydrostatic head beneath the top stratum at the time sand boils first appear. In this method i_c is determined from the following formula:

$$i_c = \frac{h_{x(c)}}{z_t} \tag{18}$$

Critical gradients as measured in the field at some of the piezometer sites are subsequently discussed.

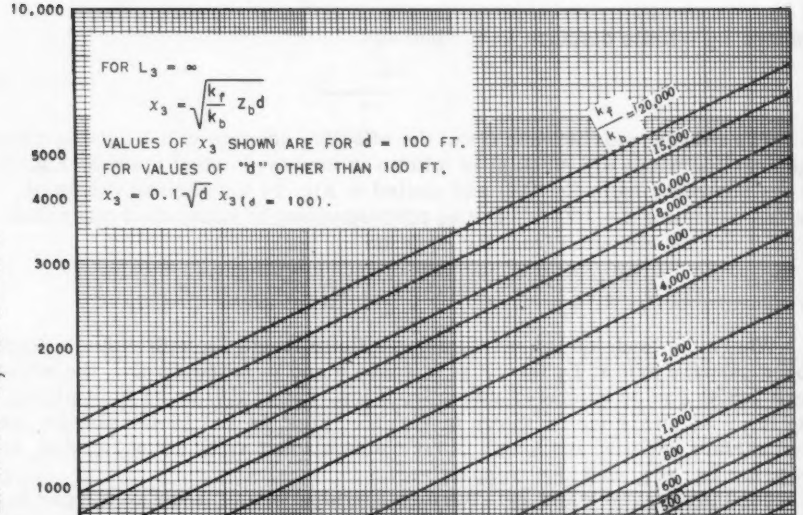


Fig. 14. Effective seepage exit length for $L_3 = \infty$ and d = 100 ft

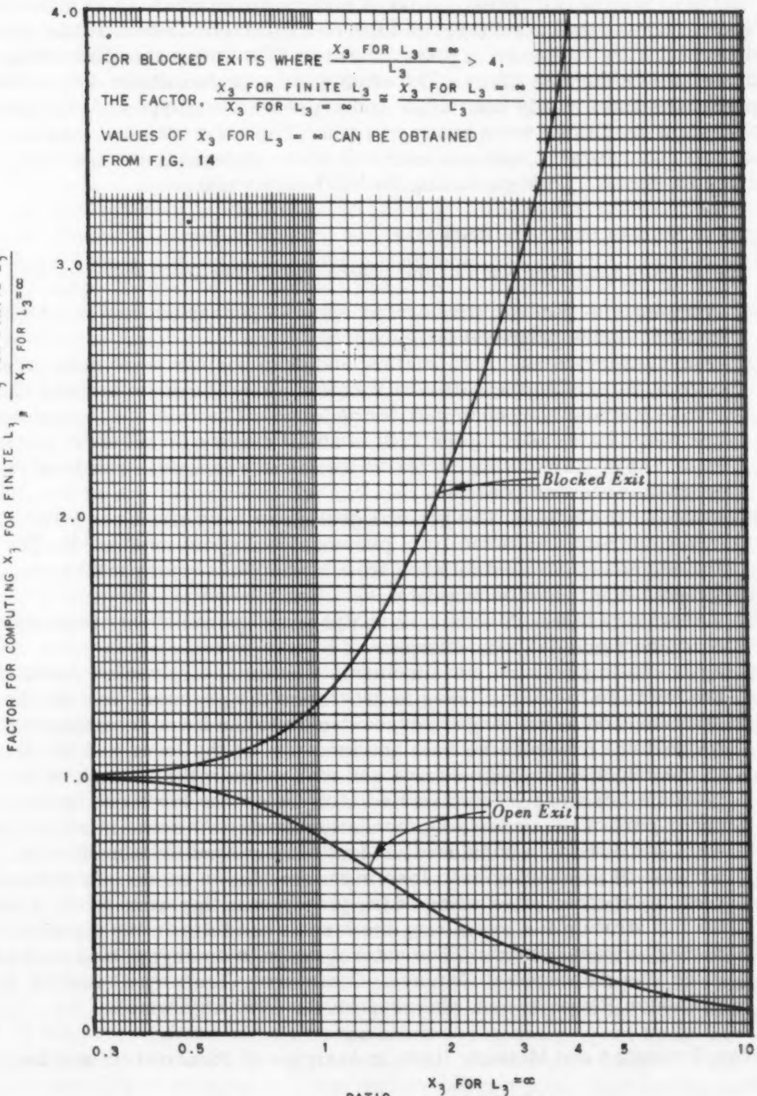


Fig. 15. Ratio between x_3 for blocked or open exits and x_3 for $L_3 = \infty$

Investigation of Underseepage at Piezometer Sites

Basic to the overall investigation of underseepage were the detailed study of geological and soil conditions and the measurement of substratum pressures and seepage flow by means of piezometers at 15 sites on the Mississippi River and one site on the Red River. The sites were selected where subsurface explorations had already been made and representative types of geological and top stratum conditions were known to exist. The sites included locations where no underseepage had occurred and where underseepage and sand boils had been a serious problem during the 1937 high water.

Types of Studies Made for Each Site

The studies made for each of the piezometer sites included:

a. Mapping. In general, three maps—plan, topographic, and an aerial mosaic—were prepared for each site.
b. Field explorations. Both shallow and deep borings were made at each site to determine the soil stratification along the levee toe and along cross sections perpendicular to the levee. The rate of seepage was determined by measuring the flow in small ditches or culverts by means of a midget Gurley flow meter at locations where delineation of the seepage area was possible.
c. Geological studies. The general geology of each site as regards former river courses was taken from previous geological studies.(4) The detailed geology was established from a study of aerial photographs, topographic maps, and boring data.
d. Laboratory tests. Mechanical analyses and permeability tests were made on remolded sand samples.
e. Hydrostatic pressure measurements. Piezometers were located along the landside toe of the levee to determine the pressure beneath the top stratum, and along ranges perpendicular to the levee to measure the hydrostatic pressure beneath and landward of the levee and the distances to the effective seepage source and exit. Generally the tops of the piezometers were located immediately below the top stratum in the upper part of the underlying foundation sands. At some sites the tips were put down to a considerable depth in the sand stratum for the purpose of measuring head loss in the foundation sands in a vertical direction; at other sites some of the piezometer tips were located within the top stratum for measuring the loss in head within the top stratum.
f. Seepage measurements. The natural seepage emerging landward of the levee was measured at Gammon, Commerce, Trotters 51 and 54, Stoval, and Baton Rouge sites at the crest of the 1950 high water.

Factors Evaluated and Methods Used in Analysis of Piezometric and Seepage Data

Factors L_1, L_2, and L_3 were obtained at each site from field surveys and existing maps. However, at some sites the value of L_3 was determined from a consideration of both geological information and piezometric gradient lines. The depth and permeability of the pervious foundation, and thickness and permeability of the top stratum at the sites, were estimated from field explorations, pumping tests, laboratory tests, and analysis of piezometric and

seepage observations. The distance to the effective seepage entrance and exit, substratum pressures landward of the levee, and hydraulic gradients beneath and landward of the levee were determined from piezometric data. Seepage flow beneath the levee was estimated from hydraulic gradients and characteristics of the pervious substratum.

River stages during significant high-water periods were obtained from gages installed at the sites. Gages and piezometers were read at two- to three-day intervals during significant high-water periods.

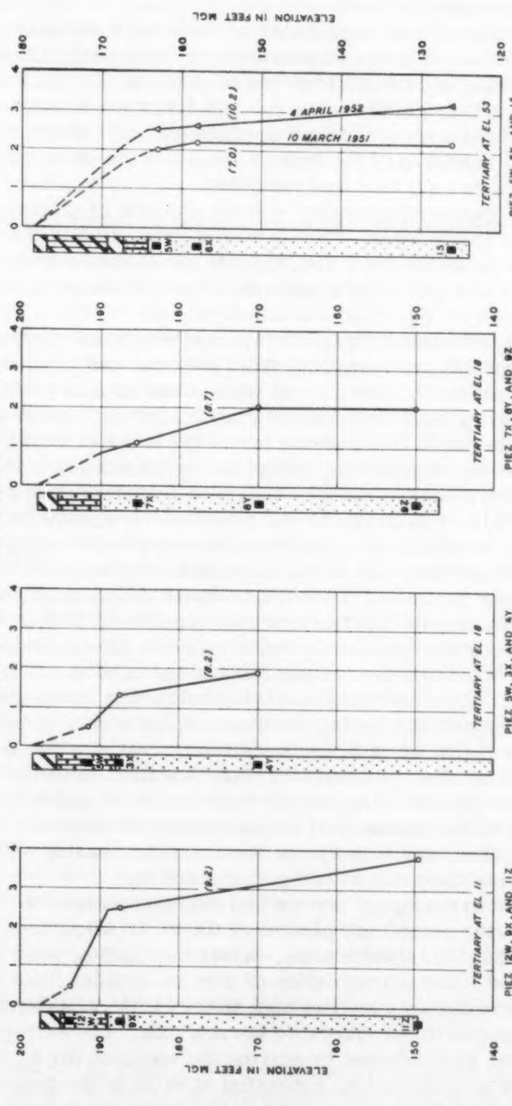
Where the ground landward of the levee is essentially level, tailwater was assumed equal to about the average elevation of the ground. Water was ponded over a part of the area landward of the levee at some of the sites and in those cases the elevation of water surface was recorded.

Seepage Source and Exit.—At each site at least one line of piezometers was installed perpendicular to the levee. The hydraulic grade line in the substratum sands and the distances from the landside toe of the levee to the effective seepage source and exit were determined from readings of these piezometers.

The distances to the effective seepage source and exit were computed from Eqs. (10) and (13), respectively, using data from piezometers located beneath the levee (and berm if present). When there were three or four piezometers beneath the levee, s and x_3 were determined graphically. Unless otherwise noted, s and x_3 are referred to the landside toe of the levee or berm.

Computation of s and x_3 requires the use of the average head in the pervious substratum at the points of measurement. Only at a few sites such as Commerce, Trotters 51, and Trotters 54 were piezometers installed sufficiently deep in the pervious foundation to obtain the average pressure in the sands. Data at these sites indicate that: (a) the head immediately beneath the top stratum under the middle portion of the levee is equal to that at any depth in the pervious substratum, and (b) where there is a significant flow of upward seepage the head immediately beneath the top stratum at the landside toe of the levee is somewhat less than the average head in the sand stratum at the levee toe (see Fig. 16). The difference in head developed at the landside toe of the levee immediately beneath the top stratum and at or near the mid-depth of the substratum sand at the above sites is plotted for various net heads in Fig. 17. At sites where no deep piezometers were installed and where the line consisted of two shallow piezometers, one beneath the levee and one at the levee toe (or berm toe where present), it was necessary to estimate the average head in the substratum sand at the levee toe from the reading of the shallow piezometer at the toe prior to computing s and x_3.

Distances to the effective seepage source and exit determined on selected days during the high-water period are plotted vs the river stage occurring on that day, e.g., Fig. 18-20. From such plots, values of s and x_3 were extrapolated to the project flood. The extrapolation of s to the project flood was based on observed trends during previous high waters with consideration given to the possibility of changes in the riverside borrow pits. The extrapolation of x_3 to the project flood was obtained by solving the equation for h_o on Fig. 7 for x_3, using the values of s, H, and h_o estimated to exist at the project flood. In so doing, h_o was taken as the average head in the substratum sand at the landside toe of the levee. A curve then was drawn through the observed seepage exits (plotted against the corresponding river stages) to the extrapolated x_3 at the project flood.



Commerce, Miss.

Trotters 54, Miss.

Fig. 16. Piezometric head above ground surface at various depths landward of levee. (Note: figures in parentheses are river stage H above ground on 1 February 1950)

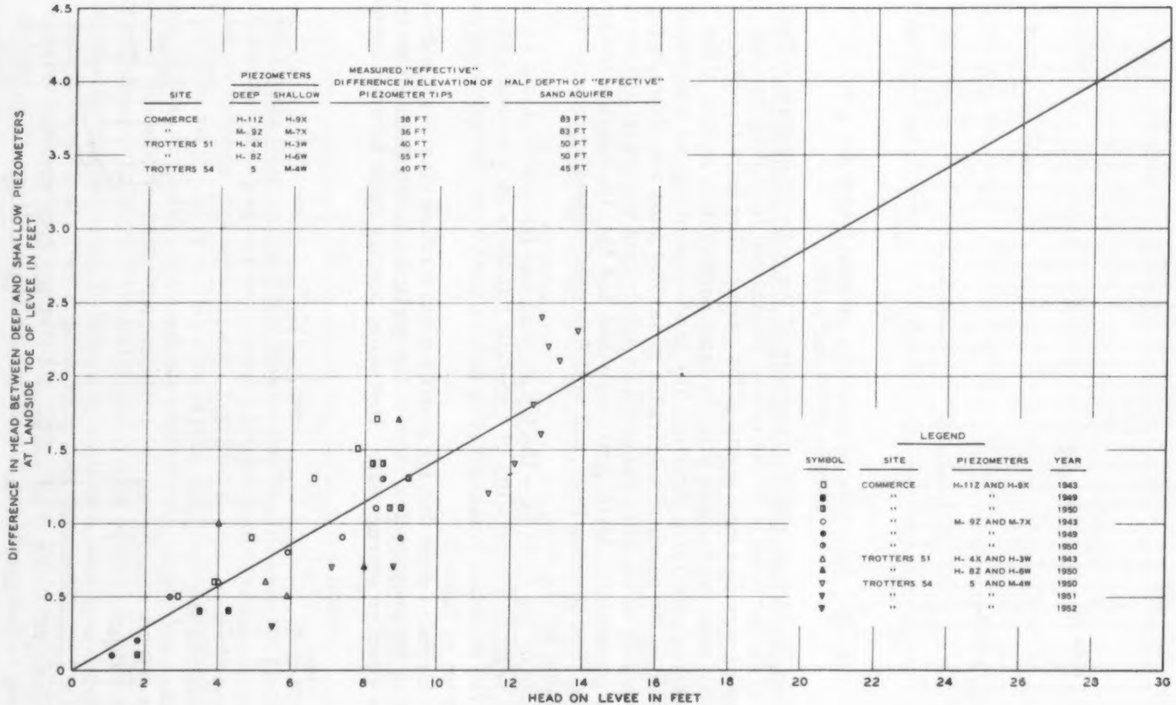


Fig. 17. Difference in head at deep and shallow piezometers at landside toe of levee

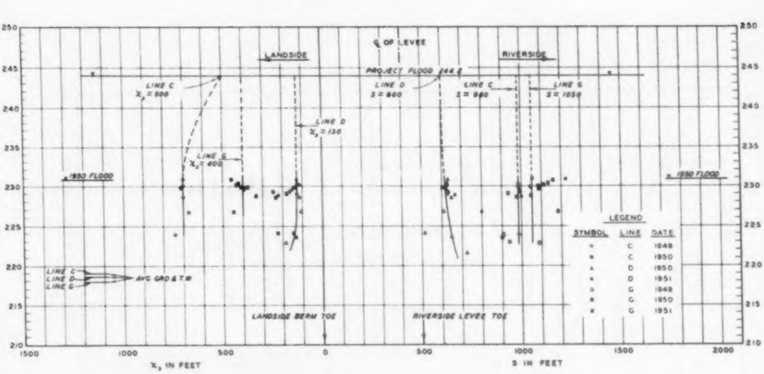


Fig. 18. Distances to effective seepage source and exit. Gammon, lines C, D, and G

Thickness, Gradation, and Permeability of Pervious Substratum.—The total thickness of the pervious substratum was determined from deep borings. The thicknesses of very fine sand strata of low permeability were transformed into reduced equivalent thicknesses with the same permeability as that of the principal seepage-carrying stratum. The gradations of typical samples of sand taken from the pervious substratum at various sites are shown in Fig. 21. The permeability of the effective pervious substratum was estimated from laboratory permeability tests, grain-size data and Fig. 12, seepage and piezometric data, field pumping tests, and well flow data, depending on what data were available for the site.

Thickness of Top Stratum.—The top stratum was transformed into a blanket of uniform vertical permeability of a specific thickness as previously described. The thickness transformation factors used were approximately the same as given in Table 2.

Blanket formulas show that for a uniform top stratum infinite in landward extent 64% of the seepage flow rises to the surface between the landside levee toe and the effective seepage exit. Therefore, borings made between the toe

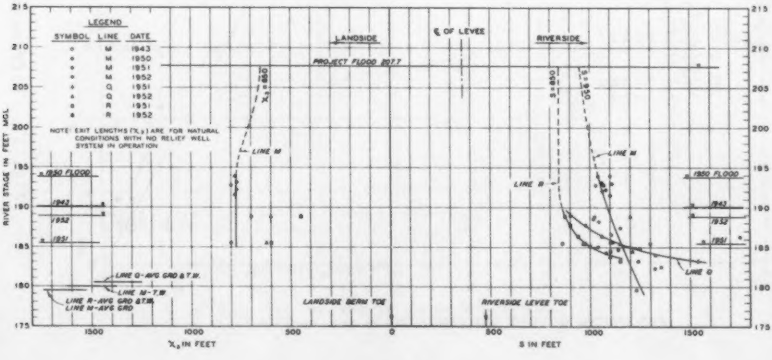


Fig. 19. Distances to effective seepage source and exit. Trotters 54, lines M, Q, and R

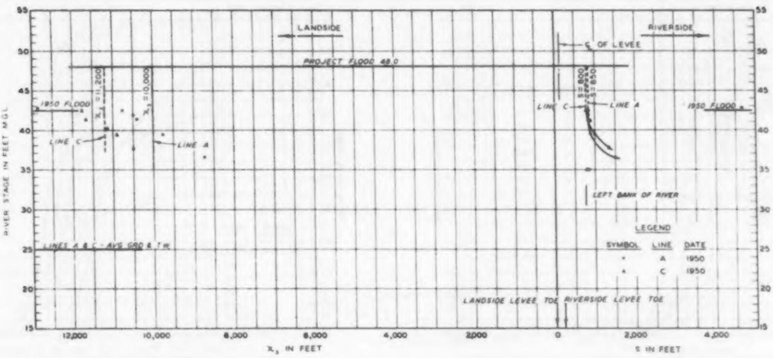


Fig. 20. Distances to effective seepage source and exit. Baton Rouge, lines A and C

of the levee and the effective seepage exit were given more weight than those farther landward when estimating z_{bL}.

Permeability of Top Stratum.—The permeability of the top stratum k_{bL} was estimated by means of blanket formulas and measurements of natural seepage.

Permeability Ratio k_f/k_{bL}.—The ratio of the permeability of sand substratum to that of the top stratum was also estimated from blanket formulas and by taking the ratio of estimated values of k_f and k_{bL}.

Seepage Beneath Levee Q_s.—The seepage beneath the levee per 100 ft of levee was computed for the crest of the 1950 flood and was also computed for the project flood from equation for Q_s on Fig. 7 using the best estimated values of d, k_f, s, and x_3 for the project flood (see Table 3). The severity of seepage at the sites during the 1950 high water was based on the following arbitrary classification.

Q_s/H gpm per 100 ft of Levee	Severity of Seepage
>10	Heavy
5-10	Medium
<5	Light

Piezometer Readings vs River Stage.—Readings of selected piezometers at the landside toe of levee or berm were plotted against the corresponding river stages for different high waters, e.g., Fig. 22-24. The ratio of the head h_o at the different piezometers to the net head H on the levee was computed and is shown on the curves drawn through the plotted data. Also shown are the estimated hydrostatic heads for the project flood; these heads were based on extrapolations of observed data and consideration of the computed maximum possible substratum heads. Values of h_c were computed by multiplying the transformed thickness of top stratum z_t by 0.85 (assumed i_c). The maximum piezometer readings were computed by adding h_c to either the ground elevation at the piezometer or the tailwater elevation where the ground at the piezometer was submerged.

The hydrostatic head h_o at various piezometers landward of the levee was estimated for the project flood as described below.

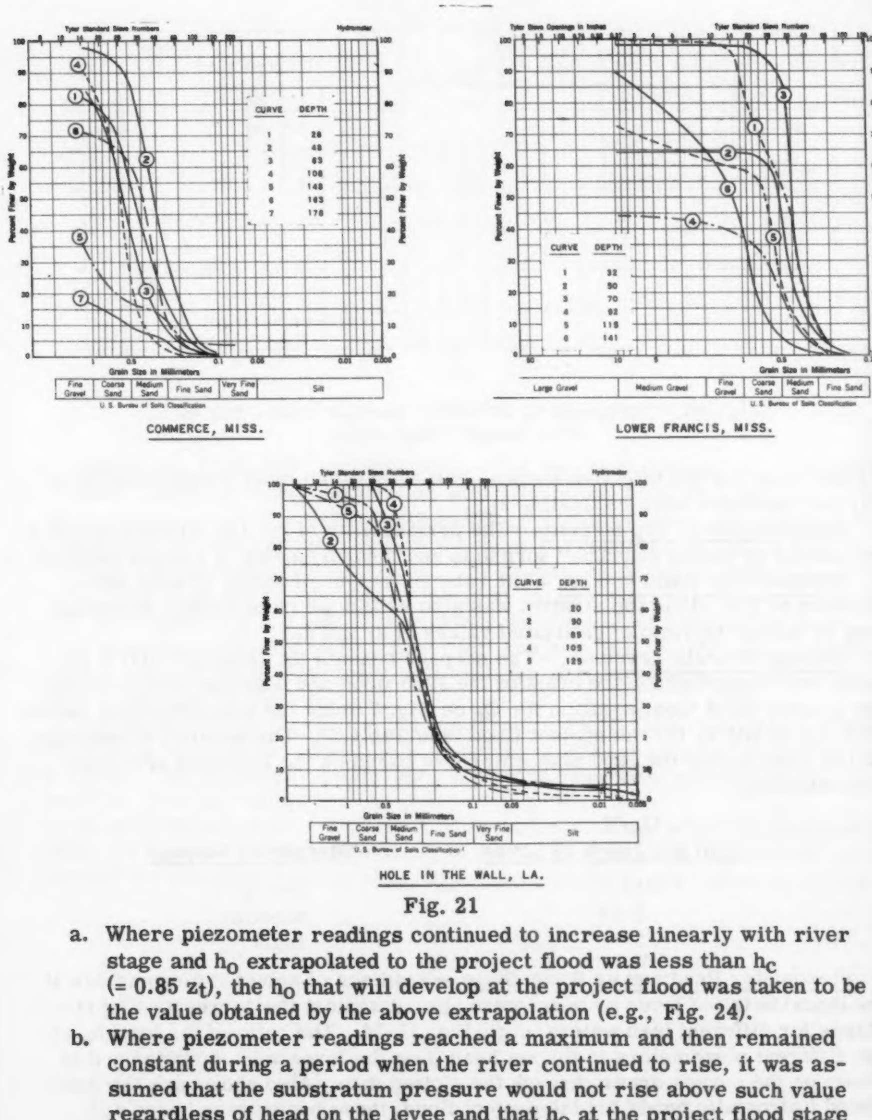


Fig. 21

a. Where piezometer readings continued to increase linearly with river stage and h_o extrapolated to the project flood was less than h_c ($= 0.85\ z_t$), the h_o that will develop at the project flood was taken to be the value obtained by the above extrapolation (e.g., Fig. 24).
b. Where piezometer readings reached a maximum and then remained constant during a period when the river continued to rise, it was assumed that the substratum pressure would not rise above such value regardless of head on the levee and that h_o at the project flood stage would be the same as the above observed maximum (e.g., Fig. 22).
c. Where piezometer readings increased with rising river stages but it appeared h_o would become equal to h_c before the project flood stage was reached, the curve fitting the data was extrapolated to $h_o = h_c$ (for $i = 0.85$); e.g., see Fig. 23.

Landside piezometer readings remaining constant over a period when the river stage continued to rise indicated that the critical head beneath the top stratum had developed and that sand boils, with severity dependent on the

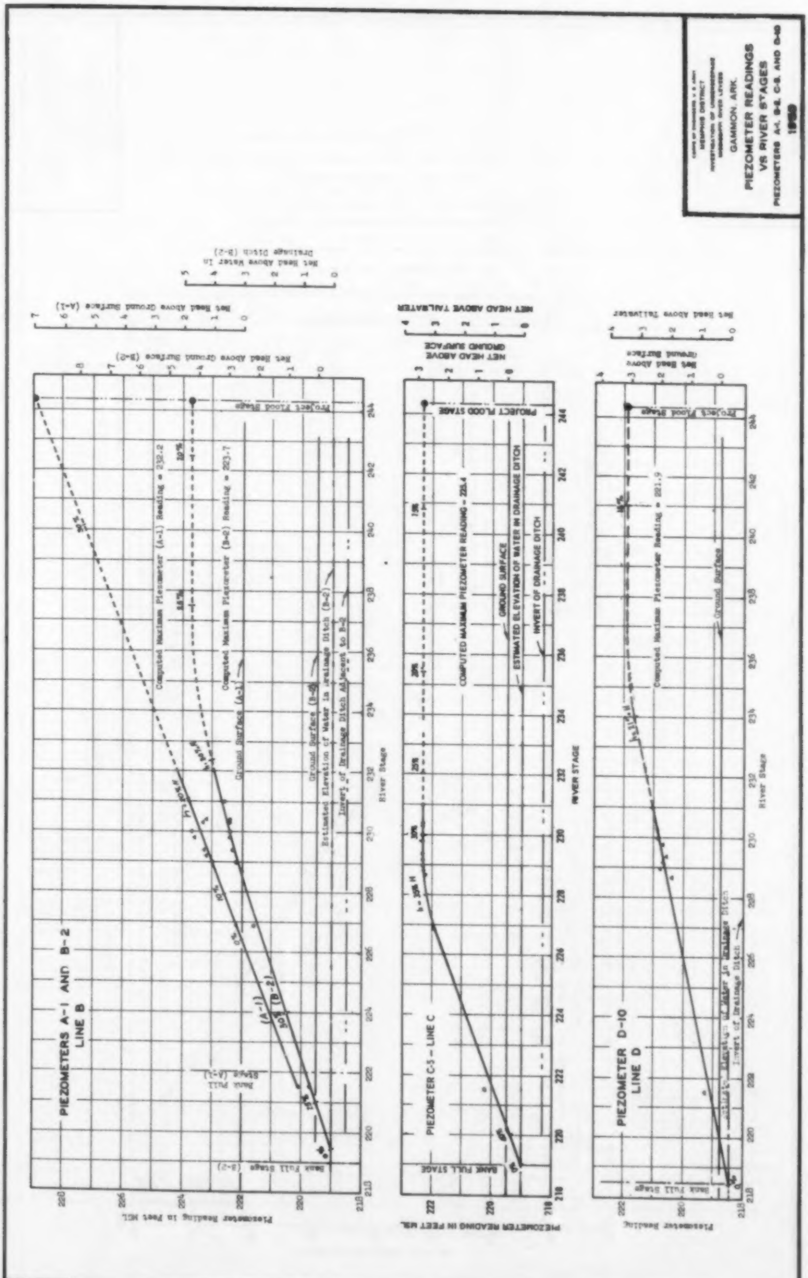

GAMMON, ARK.
PIEZOMETER READINGS
VS RIVER STAGES
PIEZOMETERS A-1, B-2, C-5, AND D-10
PIEZOMETERS A-1 AND B-2
LINE B
PIEZOMETER C-5 – LINE C
PIEZOMETER D-10
LINE D
River Stage
RIVER STAGE
PROJECT FLOOD STAGE
BANK FULL STAGE

Fig. 22

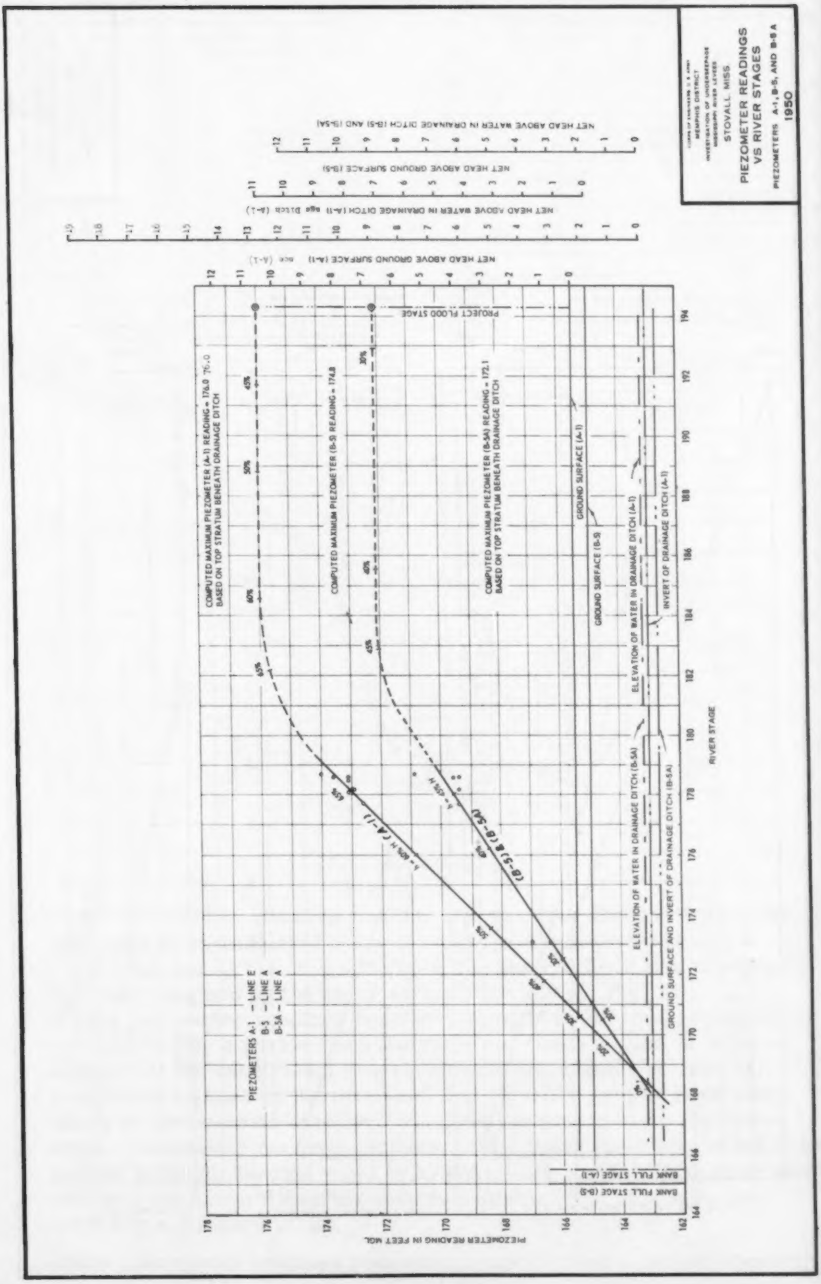

STOVALL, MISS.
PIEZOMETER READINGS VS RIVER STAGES
PIEZOMETERS A-1, B-5, AND B-5A
1950
PIEZOMETERS A-1 – LINE E
B-5 – LINE A
B-5A – LINE A
COMPUTED MAXIMUM PIEZOMETER (A-1) READING = 176.0 BASED ON TOP STRATUM BENEATH DRAINAGE DITCH
COMPUTED MAXIMUM PIEZOMETER (B-5) READING = 174.8
COMPUTED MAXIMUM PIEZOMETER (B-5A) READING = 172.1 BASED ON TOP STRATUM BENEATH DRAINAGE DITCH
PROJECT FLOOD STAGE
GROUND SURFACE (A-1)
GROUND SURFACE (B-5)
ELEVATION OF WATER IN DRAINAGE DITCH (A-1)
INVERT OF DRAINAGE DITCH (A-1)
ELEVATION OF WATER IN DRAINAGE DITCH (B-5A)
GROUND SURFACE AND INVERT OF DRAINAGE DITCH (B-5A)
BANK FULL STAGE (A-1)
BANK FULL STAGE (B-5)
RIVER STAGE
PIEZOMETER READING IN FEET MSL
NET HEAD ABOVE GROUND SURFACE (A-1)
NET HEAD ABOVE WATER IN DRAINAGE DITCH (A-1)
NET HEAD ABOVE GROUND SURFACE (B-5)
NET HEAD ABOVE WATER IN DRAINAGE DITCH (B-5) AND (B-5A)

Fig. 23

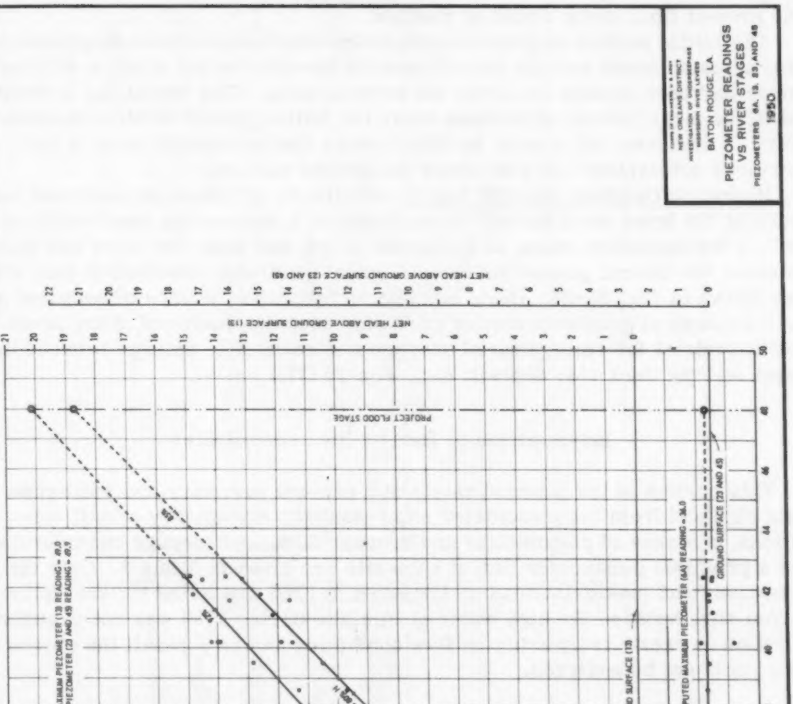

CORPS OF ENGINEERS U.S. ARMY
NEW ORLEANS DISTRICT
INVESTIGATION OF UNDERSEEPAGE
MISSISSIPPI RIVER LEVEES
BATON ROUGE, LA.
PIEZOMETER READINGS
VS RIVER STAGES
PIEZOMETERS 6A, 13, 23, AND 45
1950
PIEZOMETERS 6A – LINE A
13 – LINE G
23 AND 45 – LINE C
PIEZOMETER READING IN FEET MSL
RIVER STAGE
NET HEAD ABOVE GROUND SURFACE (13)
NET HEAD ABOVE GROUND SURFACE (23 AND 45)
PROJECT FLOOD STAGE
BANK FULL STAGE (13)
BANK FULL STAGE (6A)
BANK FULL STAGE (23 AND 45)
GROUND SURFACE (13)
GROUND SURFACE (6A)
GROUND SURFACE (23 AND 45)
COMPUTED MAXIMUM PIEZOMETER (6A) READING = 24.0

Fig. 24

height of river stage above that causing h_c, had developed at the site before the project flood crest would be reached.

The initial portion of plots of piezometer readings vs river stage indicate that artesian heads usually did not develop beneath the top stratum until after water had been against the levee for several days. This initial lag is attributed to the large volume of storage above the initial ground-water table landward of the levee which must be filled before the hydrostatic head in the pervious substratum can rise above the ground surface.

Hydraulic Gradient Beneath Levee.—Hydraulic gradients beneath and landward of the levee were plotted for each site at a river stage equal to about 1/3 or 1/2 the maximum stage, at maximum stage, and when the river had fallen to about the natural ground surface. Typical gradients observed at four sites are shown in Fig. 25-28. (Note location of "effective" source of seepage; also note flatness of gradients across thick clay deposits landward of the levee which restrict the emergence of seepage and cause it to emerge between the levee and the thick clay deposit; e.g., Fig. 25-27.)

Investigation of Data at Piezometer Sites

This portion of the paper consists of a general summary and evaluation of data obtained from the piezometer sites studied. A summary of soil conditions, analyses of piezometric and seepage data, and seepage considerations for a principal piezometer line at each site are given in Table 3. Only the soil conditions and maximum head on the levee in 1950 are listed for the Cotton Bayou site, because the high water at this site during 1950 was not of sufficient duration to create truly artesian flow conditions and as a result the piezometer data could not be analyzed.

Effect of Geologic and Man-Made Features on Underseepage

Geology and Seepage.—Geologic and natural topographic features were found to affect both the distribution and concentration of seepage landward of levees and, to some extent, the magnitude of substratum pressures.

At sites where point bar deposits predominated, the heaviest seepage and sand boils occurred in ridges adjacent to swales. Higher elevation of the surface of point bar deposits landward of low ground also prevented the exit of seepage landward of the low topography because the substratum pressure was not as high as the ground surface.

At piezometer sites where the top stratum consists of wide and fairly thick channel fillings, rather high excess heads (h_o/H) developed landward of the levee during 1950; seepage emerging through the top stratum was rather uniform and fairly light for the maximum river stage that developed.

Where the levee is founded on relatively continuous silty natural levee deposits underlain by clay, some minor seepage occurred through the natural levee deposits, although to date no sand boils have been attributed to such seepage.

In general, it is believed that, if a levee has adequate base width as compared to the net head, seepage through natural levee deposits will probably not be of a serious nature.

The thickness of backswamp clay deposits usually precludes the development of serious seepage. However, if seepage has a ready entry into the pervious substratum, high substratum pressures can be expected to develop;

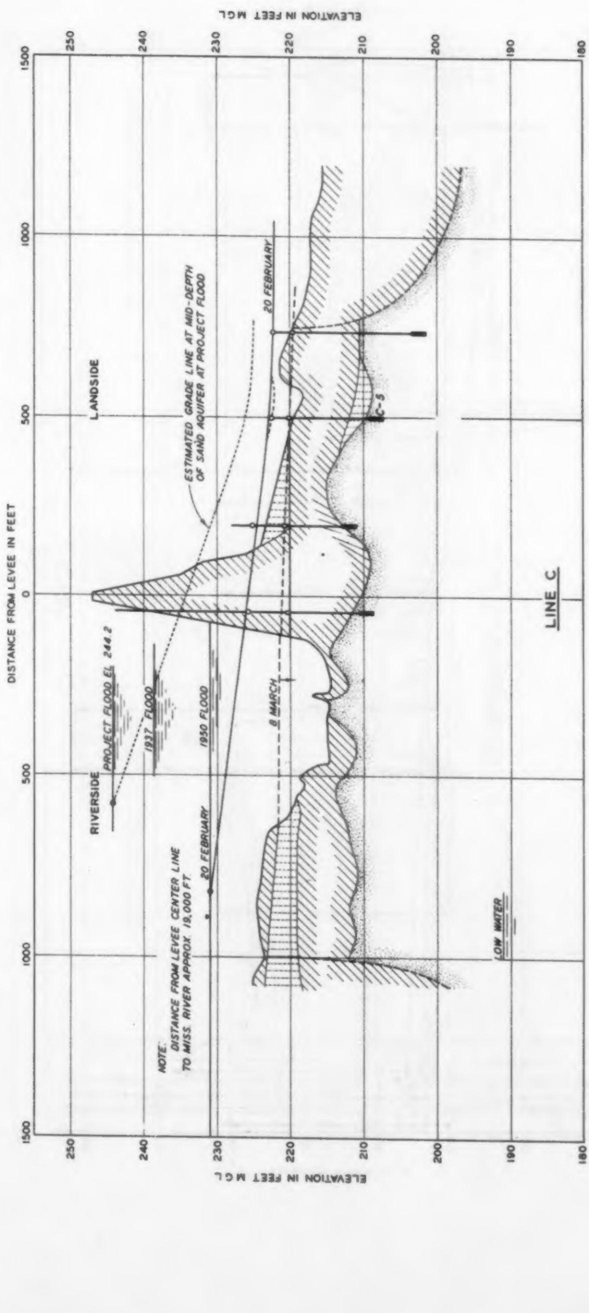


Fig. 25. Piezometric gradients, Gammon. Ark., 1950

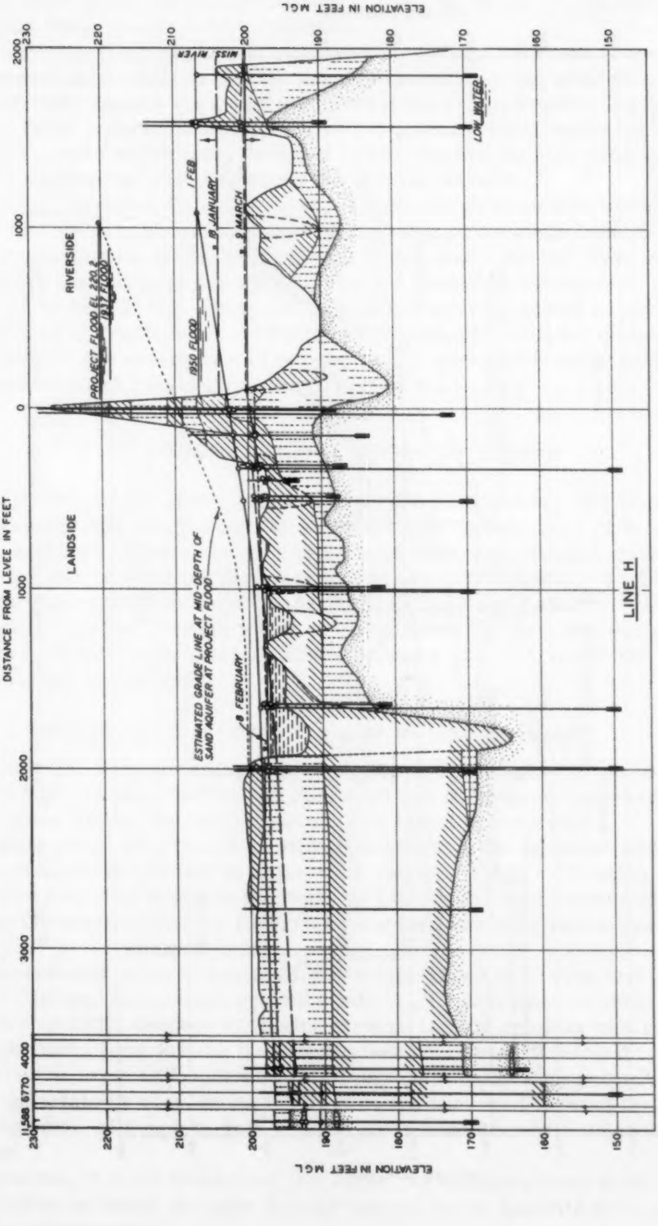


Fig. 26. Piezometric gradients, Commerce, Miss., 1950

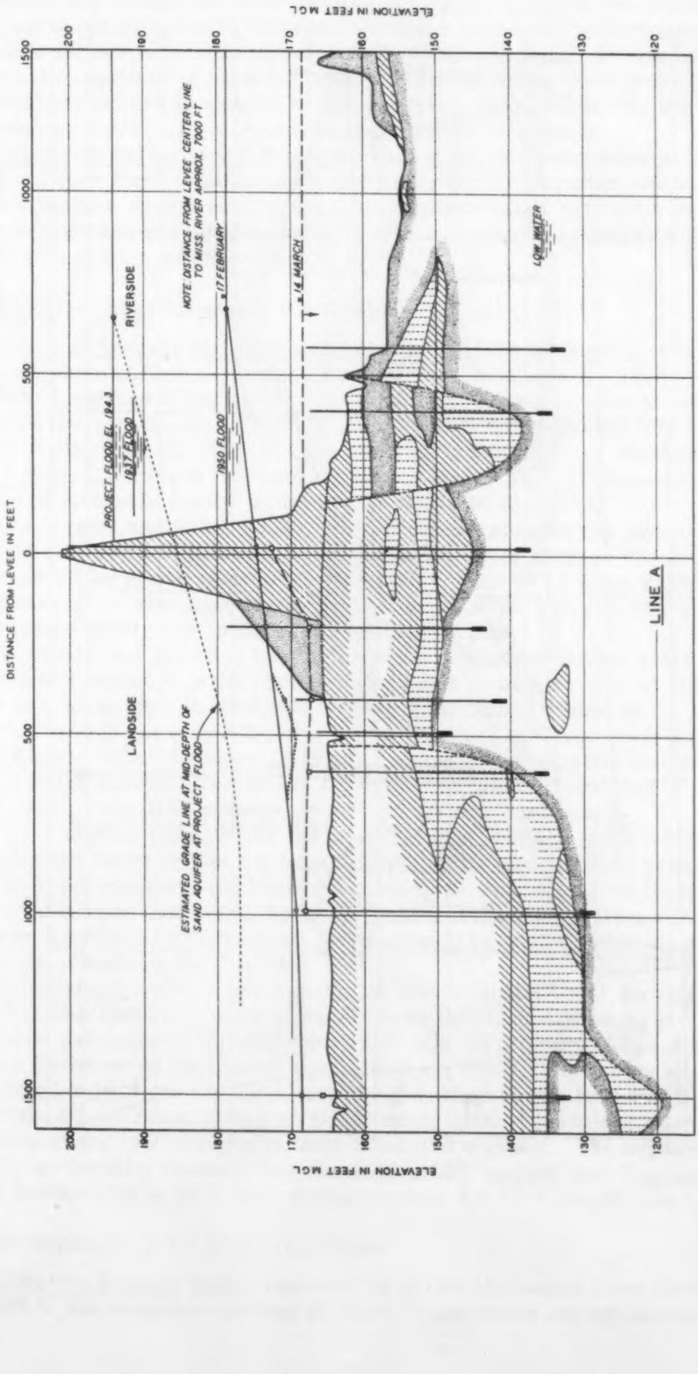


Fig. 27. Piezometric gradients, Stovall, Miss., 1950

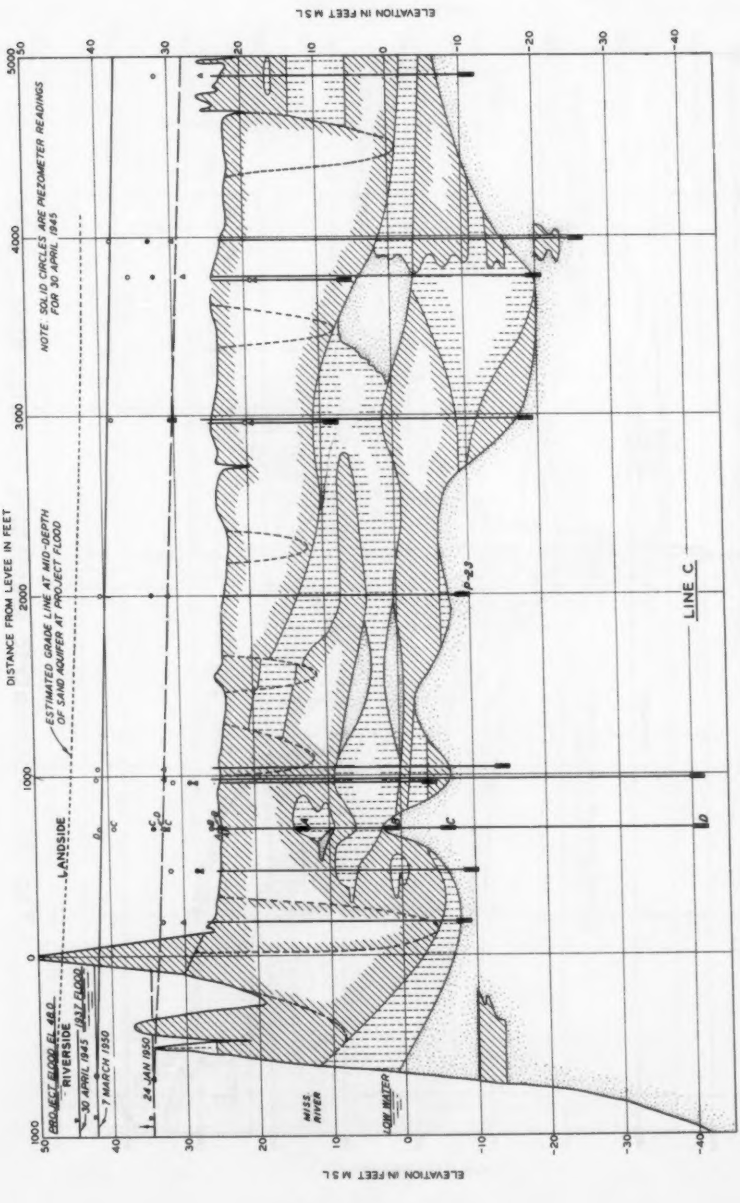


Fig. 28. Piezometric gradients, Baton Rouge, La., 1945 and 1950

and if for any reason the continuity of the clay is broken or the thickness of the clay is not adequate to withstand the uplift pressure, underseepage and possibly severe sand boils may develop. Although the top stratum at Baton Rouge is comprised of point bar deposits, the thickness is so great (30 ft) that the seepage pattern is similar to that expected along levees founded on thick backswamp clays; i.e., high excess heads and little seepage.

Man-Made Features and Seepage.—Man-made discontinuities in the top stratum were found to influence significantly the development of sand boils. Discontinuities encountered at the sites studied consist primarily of landside drainage ditches and seismic shot points, both of which appeared to be the cause of a number of sand boils.

Characteristics of Riverside Top Stratum

Source of Seepage and Effective Length of Riverside Blanket.—Of the 15 sites at which sufficient piezometric data were available for analysis, the source of seepage at the crest of the 1950 high water was located in the riverside borrow pits except where the borrow pits were blanketed with a thick layer of clay (see Fig. 25-27). The distance to the effective source of seepage entry generally ranged from about 600 to 3000 ft. The corresponding effective length of riverside blanket ranged from about 200 to 2800 ft.

At L'Argent and Baton Rouge, where the riverside borrow pits are blanketed with clay 15 to 20 ft thick, the effective source of seepage was located at the bank of the Mississippi River (Fig. 28). At the other sites where the thickness of the riverside blanket ranged from about 0 to 5 ft, the source of seepage generally was in the riverside borrow pits.

Generally, the effective length of riverside blankets tended to increase as the blanket material in the borrow pits graded from silty sand to clay, and also as a given type of blanket increased in thickness (Table 4). A top stratum of silty sand is not very effective, as the length of this type riverside top stratum was about the same as at sites where the substratum sands were exposed in the borrow pits. Where the riverside blanket consisted of clay 10 to 15 ft thick, very little seepage penetrated through the blanket.

These studies indicate that the underseepage problem along the Lower Mississippi River levees has been aggravated by more or less complete removal of the riverside top blanket along certain reaches of the levee as the result of borrow operations for construction of the levee. Where feasible, borrow operations riverward of a levee should be controlled so as not to expose the underlying sand aquifer.

Permeability.—For a given material, the permeability of the riverside top stratum k_{bR} generally tends to decrease as the thickness of top stratum increases, particularly for clay top strata. Values of k_{bR} were zero at sites where thickness of the blanket equalled or exceeded 15 ft of clay as compared to about 1×10^{-4} cm per sec where the clay blanket was less than 5 ft thick. No apparent decrease in k_{bR} with increasing blanket thickness was observed at sites where the borrow pits were blanketed with silt. The average permeability of the silty blankets was about 2.5×10^{-4} cm per sec; k_{bR} for silty sand blankets up to 10 ft thick averaged about 6×10^{-4} cm per sec.

Characteristics of Landside Top Strata

Effective Seepage Exit.—Values of x_3 generally ranged from about 150 to 11,000 ft, the largest occurring at Baton Rouge where the top stratum is about

Table 3

Summary of 1950 High-water Data at Piezometer Sites

Site	Station	Line	H 1950	H Project Flood	L_1 ft	L_2 ft	L_3 ft	Foundation d ft	Foundation $k_f \times 10^{-4}$ cm/sec	Top Stratum Soil Type*	Top Stratum z_{bL} ft*	Top Stratum $k_{bL} \times 10^{-4}$ cm/sec
C'ville[a]	26/0+00	A	9.4	19.2	1,200	330	∞	100	1,500	Cl-Si	6.5	17
Gammon	138/26+00	C	11.9	25.2	20,000	500	300	130	1,000	Clay	7.5	5
Commerce	23/10+75	H	9.2	22.7	1,800	400	1,400	165	1,000	Cl-Si Sd	7.0	2.0
Trot 51[b]	52/22+00	H	9.0	23.7	3,100	450	1,300	100	1,000	Cl-Sd Si	5.0	1.0
Trot 54[b]	54/1+05	M	13.8	27.7	2,500	475	∞	90	1,250	Clay	9.0	1.4
Stovall	77/38+00	B	14.9	29.8	3,000	600	900	40	2,500	Si-Cl	15.0	4.0
Farrell	81/24+00	A†	6.8	24.1	5,000	500	∞	20	300	Clay	4.0	1.22
		A††	6.8	24.1	5,000	500	∞	70	1,000	Cl & Sd	6.0	0.49
U Fran[c]	39+00	B	8.3	25.0	6,800	450	3,500	125	1,400		18.0	1.6
L Fran[c]	130+00	C	13.6	28.6	1,100	575	850	135	1,600	Clay	5.0	17
Bolivar	2199+75	D	6.5	26.2	1,500	330	900	90	1,200	Clay	13.0	10
Eutaw	2860+00	D	6.2	28.1	2,500	450	∞	70	1,100	Sd Si	18.0	1.3
L'Argent	3542+33	B	16.4	31.0	2,500	380	1,340	120	400	Clay	15.5	0.24
H W[d]	3618+95	D	10.4	24.5	2,100	500	∞	130	500	Cl-Sd Si	4.0	0.72
Kelson	2700+71	B	16.7	21.7	1,000	180	400	6‡‡	5‡‡	Si Cl	2.0	0.025
Baton R[e]	97+42	C	17.4	23.0	500	210	5,000	175	500	Si Cl	30.0	0.06
Cotton[f]	826+49	B	4.5	17.0§§	1,050	200	2,300	30	300	Clay	7.5	------

[a] Caruthersville.
[b] Trotters 51 and Trotters 54.
[c] Upper Francis and Lower Francis.
[d] Hole-in-the-Wall.
[e] Baton Rouge.
[f] Cotton Bayou.
* Soil types and thickness are approximate; both may vary considerably along the piezometer line.
** Q_A and Q_A/H are measured seepage flows in limited areas landward of levee and do not represent the tot
† Upper aquifer.
†† Lower aquifer.
‡ Near high bank of Mississippi River.
‡‡ Values for natural levee deposit in which piezometers were installed.
§ Type and thickness of blanket over deep pervious sand; shallow natural levee stratum of silt in which pi
§§ Interim flood.

30 ft thick. The distance to the effective seepage exit was usually rather short at sites where the landside top stratum was thin and at sites where numerous sand boils developed; it was relatively long where the top stratum was thick or the exit of seepage was partially blocked as a result of landward swales or sloughs.

Values of x_3 followed three basic patterns during rising river stages: (a) a constant x_3 indicated that resistance to the flow of seepage either landward or up through the natural blanket was constant for the river stages experienced; (b) a decrease in x_3 with rising river stage usually occurred when sand boils began to develop (such boils provide additional seepage outlets, thereby decreasing the resistance offered by the natural blanket to the emergence of seepage), (c) an increase in x_3 with rising river stage indicated an increase in resistance to the flow of seepage landward. (At the beginning of overbank stages, the natural water table may be low and seepage may readily flow into the resultant large volume of ground-water storage which in a sense acts as a drainage area. As the subsurface storage becomes filled, the phreatic line comes in contact with the bottom of the top stratum and seepage either has to flow toward storage areas farther landward or force its way up through the top stratum. In either case resistance to the flow of seepage landward is increased, thereby increasing the distance to the effective seepage exit.)

Table 3

(Continued)

at Piezometer Sites

k_f/k_{bL}	Obs x_1 ft	S ft	x_3 ft	x_3, ft at River Stage at Which $i = 0.85$	h_o, ft 1950	h_o, ft Project Flood	h_o (max), ft Computed from $i_c = 0.85$	h_o (max), ft Observed	$\frac{h_o}{H}$ % 1950	Natural Seepage in gpm/100 ft of Levee (1950) Q_s	Q_s/H	Q_A**	Q_A/H**
90	230	560	240	240	2.0	3.2	3.2	2.0	21	260	28.0	-----	-----
200	480	980	700	700	3.3	3.3	6.4	3.3	28	135	11.3	71.4	6.0
500	1,100	1,500	1,000	950	2.2	3.2	3.2	2.2	25	91	9.9	60.7	6.6
1,000	650	1,100	735	735	3.0	6.0	6.0	3.0	33	72.5	8.1	-----	-----
730	575	1,050	770	770	3.0	3.0	7.7	3.0	22	125	9.1	55.2	4.0
700	200	800	800	800	6.0	9.8	9.8	6.0	44	150	10.0	120.7	8.1
245	80	580	140	140	2.4	2.4	2.4	2.4	28	8.2	1.2	-----	-----
2,050	360	860	930	1,050	6.2	17.5	20.0	6.2	72	37.5	5.5	-----	-----
870	1,070	1,520	1,400	2,150	1.5	15.3	15.3	1.5	21	73	8.8	-----	-----
95	435	1,010	250	250	1.7	3.8	3.8	1.7	13	340	25.2	-----	-----
115	320	650	365	365	2.4	4.2	11.0	2.4	37	101	15.6	-----	-----
875	1,150	1,600	1,050	1,050	6.0	11.0	11.0	6.0	65	27	4.3	-----	-----
1,700	2,770	3,150*	3,000	5,800	5.4	14.9	14.9	5.4	35	18	1.1	-----	-----
690	1,600	2,100	600	620	1.2	1.9	1.9	1.2	13	40	3.5	-----	-----
200**	70	250	50**	50	4.6	8.9	18.7	4.6	28	0.25	0.015	-----	-----
9,000	590	800*	11,200	11,200	12.4	15.2	26.8	12.4	73	18.7	1.1	9.05	0.52
-----	-----	-----	-----	-----	----	----	----	----	--	-----	-----	-----	-----

g beneath the levee.

talled is exposed in riverside borrow pit.

The accuracy of x_3 as determined from piezometer readings is affected by the average ground or tailwater elevation, and therefore the elevation of water in submerged areas should be determined during high water periods.

Thickness and Permeability.—The thickness of the landside top stratum varied from about 4 to 30 ft; the permeability k_{bL} generally ranged from about 0.06×10^{-4} cm per sec to about 10×10^{-4} cm per sec (Tables 3 and 5). At sites where numerous sand boils occurred, considerably higher values of permeability were noted. Most values of k_{bL} at the crest of the 1950 high water ranged from about 0.5 to 10×10^{-4} cm per sec.

There was a pronounced trend for k_{bL} to decrease with an increase in z_{bL}, particularly for clayey top strata; there was a lesser tendency for k_{bL} to decrease with increasing thickness of silty top stratum. The permeability of top strata less than 10 ft thick was about the same for silt as for clay. The permeability of the top stratum landward of the levee has little relation to that which would be obtained from laboratory tests on undisturbed samples, but instead depends to a large extent on the presence and numbers of fissures, root holes, former boil holes, and other perforations in the top stratum. The effect of these perforations in clay top stratum appears to be reduced if the blanket thickness exceeds 10 ft, and greatly reduced if z_{bL} exceeds 15 ft.

Table 4

Summary of Distances to Effective Source of Seepage, Effective Lengths of Riverside Blankets, and Vertical Permeability of Riverside Blanket Materials at the Crest of 1950 High Water

Blanket in Riverside Borrow Pit		Number of Piezometer Lines from Which Data Were Obtained	s, ft			x_1, ft*			$k_{bR} \times 10^{-4}$ cm/sec			Suggested Design Values	
Soil Type	Thickness in ft		Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	k_{bR}	x_1
Sand	---	3	1080	800	960	480	200	370	----	----	----	----	250
Silty Sand**	<5	3	800	560	670	320	230	280	14.2	1.6	7.0	7.0	300
	5 to 10	1	560	560	560	280	280	280	1.8	1.8	1.8	2.5	600
											5.7†		
Silt & sandy silt	<5	4	1500	600	1050	1220	270	670	7.4	0.24	2.2	2.0	400
	5 to 10	2	1600	910	1260	1190	510	850	5.0	0.33	2.7	1.5	800
											2.4†		
	>10											1.0	1200
Clay	<5	6	1280	610	1020	750	110	690	1.7	0.34	0.79	0.8	600
	5 to 10	2	1720	1520	1620	1270	1070	1170	1.3††	0.86††	1.08††	0.5	1300
	10 to 15	0	----	----	----	----	----	----	----	----	----	0.2	2500
	>15	3	3150	800	1600	∞	∞	∞	0.0	0.00	0.00	0.05	4000 or L_1‡
											0.4†,‡‡		

* Values of x_1 computed from observed values of x_1 and adjusted to a condition where $L_1 = \infty$.

** Does not include Hole-in-the-Wall where values of s and x_1 may not be reliable because artesian flow conditions did not develop until near the crest of the 1950 high water.

† Averages of all values of k_{bR} for a given soil type without regard to thickness.

†† Values are considered to be too high as at these piezometer lines (Upper Francis) seepage could enter the pervious substratum through a silty blanket riverward of the borrow pit as well as through the clay in the borrow pit.

‡ Use the smaller of the two values.

‡‡ Average does not include k_{bR} for blanket thickness between 5 and 10 feet.

Comparisons between k_{bR} and k_{bL} for similar blankets of similar thickness indicate that the landside blanket tends to be about two to ten times as pervious as the riverside blanket. As cracks and fissures exist on both sides of the levee, this difference is attributed to the tendency of upward seepage landside to flush out the cracks and perforations, thereby increasing the overall permeability of the top stratum. Downward seepage through the riverside blanket tends to seal any cracks or fissures unless excessive erosion occurs.

Characteristics of Pervious Substratum

Effective thickness of the pervious substratum ranged from about 70 to 165 ft and averaged about 110 ft for the sites studied. Estimated values of k_f ranged from 400 to 2500 x 10^{-4} cm per sec (Table 6). For most sites in the Memphis and Vicksburg Districts above L'Argent, La., k_f ranged from about 1000 to 1500 x 10^{-4} cm per sec. At L'Argent and sites farther downstream, k_f was estimated to be no more than about 500 x 10^{-4} cm per sec. Although it should not be inferred that k_f will always be less than 500 x 10^{-4} cm per sec in the alluvial valley of the Mississippi River below L'Argent, lower k_f values generally can be expected below L'Argent. Good agreement was obtained between values of k_f as estimated from a correlation of grain size and permeability and those determined from analyses of piezometric data and natural seepage measurements, well flow data, and pumping tests. Poor agreement was obtained between k as estimated from laboratory permeability tests and k_f obtained from piezometric, seepage, and well flow data, and/or pumping tests.

Ratio of Permeability of Pervious Substratum to Landside Top Stratum

Values of k_f/k_{bL} obtained at the piezometer sites at the crest of the 1950 high water ranged from about 100 to 2000 except at Baton Rouge where the ratio was about 8500 (see Fig. 29 and Table 5). There was a tendency for k_f/k_{bL} to increase for clay blankets as the top stratum increased in thickness. However, there was no apparent variation in k_f/k_{bL} with z_{bL} for sites where the top stratum was predominantly silt.

Critical Upward Gradient

Upward gradients through the top stratum as measured by piezometers during the 1950 high water and the degree of seepage were:

Seepage Conditions	i
Light to no seepage	0 to 0.5
Medium seepage	0.2 to 0.6
Heavy seepage	0.4 to 0.7
Sand boils	0.5 to 0.8

The gradient required to cause sand boils varied considerably at the different sites, possibly because at sites where sand boils had developed previously only fairly low excess heads may have been needed to reactivate these boils in 1950. At sites where no sand boils had occurred in the past, higher gradients may have been required to initiate formation of the boils, although this is difficult to ascertain because of limited data on previous seepage at the sites. From the above data, it appears that heavy seepage and sand boils should be anticipated whenever estimated upward gradients exceed 0.5 to 0.8, depending on site conditions.

Table 5

Summary of Ratios of Permeability of Pervious Substratum to Landside Top Stratum and Permeability of Landside Top Stratum at Crest of 1950 High Water

Soil Type	Thickness in ft	Number of Piezometer Lines from Which Data Were Available	k_f/k_{bL} Max	k_f/k_{bL} Min	k_f/k_{bL} Avg*	$k_{bL} \times 10^{-4}$ cm/sec Max	$k_{bL} \times 10^{-4}$ cm/sec Min	$k_{bL} \times 10^{-4}$ cm/sec Avg**	Avg k_{bL} in 10^{-4} cm/sec from x_3 at $i = i_c$	Suggested Design Values: $k_{bL} \times 10^{-4}$ cm/sec†	Suggested Design Values: k_f/k_{bL}††
Silty sand	<5	0	-----	-----	------	-----	----	----	----	10	125
	5 to 10	1	-----	-----	60	-----	----	25	25	8	150
	>10	0	-----	-----	------	-----	----	----	----	6	200
Silt and	<5	2	1,000	690	840	1.0	0.7	0.9 } 5.1	0.8 } 4.5	5	250
sandy	5 to 10	3	2,000	81	445	17	0.5	7.9	6.3	4	300
silt	10 to 15	0	-----	-----	------	-----	----	----	----	3	400
	>15	1	-----	-----	875	-----	----	1.3	1.3	2	600
Clay and	<5	3	-----	-----	95‡	17	0.03	6.1 } 8.1	6.0 } 7.9	4	250
silty	5 to 10	9	2,050	25	345	40	0.5	8.8	8.5	3	400
clay	10 to 15	4	1,270	115	640	10	1.1	4.7	4.3	1.5	800
	15 to 20	3	1,700	870	1,130	1.6	0.24	1.0	----	0.5	2,500
	>20	2	9,000	8,350	8,600	0.06	0.06	0.06	----	0.08	15,000

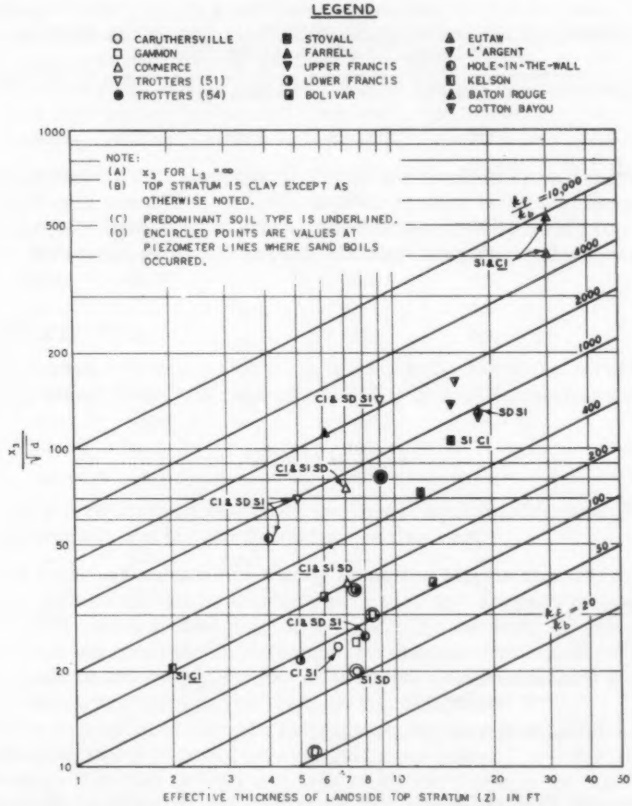
* Average = square of average of square roots of individual k_f/k_{bL} values.

** Arithmetic average.

† Values based largely on observations at the piezometer sites. They may be somewhat high for new levees and levees which have not been subjected to major high water.

†† Based on $k_f = 1250 \times 10^{-4}$ cm per sec.

‡ Value of k_f/k_{bL} for one piezometer line only. Values of k_f/k_{bL} at other two lines are not indicative of the ratio of the permeability of the entire effective pervious substratum to that of the landside top stratum because the tips of the piezometers are installed in a silty sand or fine sand stratum which is separated from the principal seepage carrying aquifer by clay strata.



g. 29. Ratio of permeability of pervious substratum to permeability of landside top stratum at crest of 1950 high water

Table 6

Summary of Coefficient of Permeability (10^{-4} cm/sec) of Pervious Substratum Obtained by Various Methods

Site	Laboratory Permeability Tests	Grain-size Data and Fig. 12	Seepage and Piezometric Data	Field Pumping Tests	Well Flow Data	Selected Value of k_f
Caruthersville	1150	----	----	----	----	1500
Gammon	750	1200	850	----	----	1000
Commerce	750	900	875*	1000	865	1000
Trotters 51	750	1000	835	----	1150	1000
Trotters 54	400	1250	1250	1180	1500**	1250
Stovall	350	950	2850†	----	----	2500
Farrell††	800	1200	----	----	----	1000
Upper Francis	900	1900	----	----	----	1400
Lower Francis	1100	2300‡	----	----	----	1600
Bolivar	----	1310	----	----	----	1200
Eutaw	----	1310	----	----	----	1100
L'Argent	----	350	----	----	----	400
Hole-in-the-Wall	60	500	----	----	----	500
Kelson‡‡	0.6	----	----	----	----	5
Baton Rouge	----	----	600§	----	----	500
Cotton Bayou	30	200	----	----	----	200

* Piezometer Line H.
** 1951 data.
† Average for piezometer lines A and B.
†† Values are for lower aquifer only.
‡ Piezometer Line C.
‡‡ Values are for upper stratum of silty sand.
§ Piezometer Line A.

Effect of Natural Partial Cutoffs and Massive Clay Deposits on Seepage

An examination of piezometric gradients where natural partial cutoffs exist shows no significant drop in head across the partial cutoffs. Massive clay deposits a short distance landward of the levee toe are believed to have increased the severity of the seepage conditions which occurred during the 1937 and 1950 high waters at Trotters 51 and Stovall.

Seepage Berms at Piezometer Sites

Except for the seepage berm at Gammon, berms at the other sites are of such soil types and/or thickness as to make them practically impervious. Assuming that the riverside and landside blankets remained unchanged as a result of construction of the berms, the 200-ft-wide berms typical of most sites probably decreased seepage and landward pressures by approximately 10 to 15 per cent from what would have occurred with no berm. Since borrow for most of these berms was obtained riverward of the levee, the borrow operations may have reduced the effective values of x_1 as much as the width of the berm increased L_2. If such is the case, little or no reduction in Q_s or h_o may have resulted from the berm. Construction of the rather thick berms at certain of the piezometer sites has practically eliminated the occurrence of

sand boils at the landside toe of the levee, and lengthened the path of any potential piping channel which would have to develop before the levee would be endangered. However, as illustrated by occurrence of a large sand boil 200 ft from the levee at Stovall during the 1937 high water, a 100- or 200-ft-wide berm does not in itself insure complete safety against underseepage.

Seepage

The computed natural seepage (Q_s/H) at the various sites ranged from about 1 to 25 gpm per 100 ft of levee depending upon source of seepage, permeability of pervious substratum, and thickness of landside top stratum (Table 3). Measured rates of seepage checked computed values reasonably well.

Hydrostatic Pressure

The hydrostatic pressure (h_o/H) at the landside toe of the levee or berm varied from about 20 to 75% depending on site and soil conditions (Table 3).

CONCLUSIONS

Data and information gained from the theoretical, model, and prototype studies furnish a basis for the following conclusions:

a. Sand boils and subsurface piping along the Mississippi River levees are the result of excess hydrostatic pressure and seepage through deep pervious strata underlying the levees. The severity of underseepage, both excess hydrostatic pressure and seepage flow, is dependent upon the head on the levee, source of seepage, perviousness of substratum, and characteristics of the landside top stratum.
b. There is a definite correlation between surface geology and the location and occurrence of underseepage and sand boils.
c. Seepage flow and hydrostatic heads landward of a levee can be estimated from seepage formulas, and/or piezometric data, and a knowledge of riverward and landward foundation characteristics.
d. Removal of the natural top blanket riverward by borrow operations has aggravated the underseepage problem along Mississippi River levees. Except where clay several feet thick was left in place, the source of seepage was in the riverside borrow pits.
e. Underseepage can be controlled by properly designed and constructed landside seepage berms, relief wells, and riverside blankets.

ACKNOWLEDGMENT

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APPENDIX A: NOTATIONS

A	Surface area in which seepage emerging landward of a levee is measured
a	Well spacing
C	Bligh's creep ratio
C_w	Lane's weighted creep ratio
c	A constant for natural top stratum where $c = \sqrt{\frac{k_b}{k_f z_b d}}$
D	Thickness of pervious substratum
D_{10}	Effective grain size, 10 per cent of grains smaller than stated size
d	Effective thickness of pervious substratum. Depth of cutoff in formulas for partial cutoffs
$d_a, d_b. \ldots d_n$	Thickness of each stratum comprising pervious substrata
G	Ratio of flow from a partially penetrating well to that from a fully penetrating artesian well
H	Total net head on levee, or height of flood stage above average low-ground surface, or tailwater, landward of levee
h	Effective (net) head acting on a line of relief wells
h_c	Maximum possible (net) head beneath top stratum; head at which upward gradient through top stratum is equal to critical gradient
h_o	Head (net) beneath top stratum at landside toe of levee (without seepage control measures) assuming top stratum capable of withstanding such a head
h_x	Head (net) beneath top stratum at distance x landward from landside toe of levee; average head beneath area A in which seepage was measured
$h_{x(c)}$	Net head above ground surface or tailwater at time sand boils or heaving of top stratum occurs
h_1, h_2	Drawdown below water table during a pumping test at distances r_1 and r_2, respectively, from test well. Substratum heads (above ground surface) at two piezometers on a line perpendicular to the levee at distances ℓ_1 and ℓ_2, respectively, from landward toe of levee
i	Upward gradient through top stratum landward of levee
i_c	Critical upward gradient through top stratum landward of levee
i_1	Allowable upward gradient at toe of landside seepage berm
i_o	Allowable upward gradient at landside toe of levee

k	Coefficient of permeability
$k_a, k_b \ldots, k_n$	Coefficient of permeability of each stratum comprising the pervious strata
k_b	Vertical permeability of top stratum
k_{bL}	Vertical permeability of top stratum landward of levee
k_{bR}	Vertical permeability of top stratum riverward of levee, particularly that in riverside borrow pits
k_f	Permeability of pervious foundation
k_H	Horizontal permeability of a pervious stratum
k_{H-n}	Horizontal permeability of individual stratum
k_v	Vertical permeability of a pervious stratum
k_{v-n}	Vertical permeability of individual stratum
L_1	Distance from riverside toe of levee to river
L_2	Base width of levee, and berm if present
L_3	Landward (effective) extent of top stratum
ℓ	Distance between two piezometers installed on a line perpendicular to the levee
ℓ_1, ℓ_2	Respective distances from landside toe of levee (or berm) to piezometers 1 and 2, installed on a line perpendicular to the levee
M	Slope of hydraulic grade line at mid-depth of pervious substratum beneath the levee
M_A	Slope of hydraulic grade line in pervious substratum at landside edge of (surface) area A
Q_A	Rate of seepage flow per unit length of levee emerging in (surface) area A
Q_s	Total seepage flow (with or without wells) per unit length of levee per unit of time
Q_w	Flow from a single relief well per unit of time
$q_a, q_b \ldots, q_n$	Flow from each stratum of the pervious strata
R	Radius of influence for a well, or maximum average rate of rainfall over an area, in inches per hour, occurring during time of concentration
r	Ratio of allowable upward gradient through top stratum at toe of levee to that at toe of seepage berm = i_o/i_1
r_w	Effective radius of a relief well
r_1, r_2	Radial distances from a test well
s	Distance from landside toe of levee (or berm) to effective source of seepage entry

W	Effective length of well screen; penetration of well screen into pervious aquifer expressed as a decimal; base width of levee in formulas for partial cutoffs
x	Distance landward from landside toe of levee
x_1	Effective length of blanket riverside of levee
x_3	Distance from landside toe of levee (or berm) to effective seepage exit
z	Total thickness of top stratum
z_{bL} or z_L	Effective thickness of top stratum landward of levee
z_{bR} or z_R	Effective thickness of top stratum riverward of levee, particularly that remaining in riverside borrow pit
z_{b-n}	Thickness of individual stratum of top strata
z_t	Critical thickness of landside top stratum
Σp	The shortest vertical path of seepage flow around a partial cutoff beneath a levee
Γ	Gamma functions(8)
\$	Shape factor, the ratio in a flow net of the number of flow channels to number of equipotential drops from the seepage source to exit

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ROCK CHARACTERISTICS AT THE PAULO AFONSO POWER PLANT[a]

Ernesto Pichler,[1] M. ASCE and Francisco Barros de Campos[2]

SUMMARY

The Paulo Afonso Project on the Rio São Francisco is the first underground project in Brazil. The conduit system, the powerhouse and the discharge system are located wholly in rock. To obtain information about the possible behavior after excavation of the rock surrounding these structures, a number of tests, "in situ" and in the laboratory, were executed. This paper presents a description of these tests and their results together with an outline of the local geology.

INTRODUCTION

The Paulo Afonso Project is located at the Paulo Afonso cataracts, near the corner common to the states of Alagoas, Pernambuco and Bahia. The project has the double purpose of flood control and power production. Structures of the project include a gravity dam, 4230 meters long, three conduit tunnels each with a diameter of 4.80 m., a length of 104.3 m. and a discharge capacity of 90 cu. m; a powerhouse 60.15 m long, 15 m. wide and 31.34 m. high, a discharge tunnel with a diameter of 10 m. and a length of 180.25 m.; control works etc.

Investigation started in 1946 and the first generator was put into operation in 1954. About 65000 cu. m. of rock were excavated. Fig. 1 presents the general plan of the project and the location of the field tests.

Note: Discussion open until January 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2137 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 4, August, 1959.

a. Presented at meeting of G.S.A., Atlantic City, N. J., November 1957.

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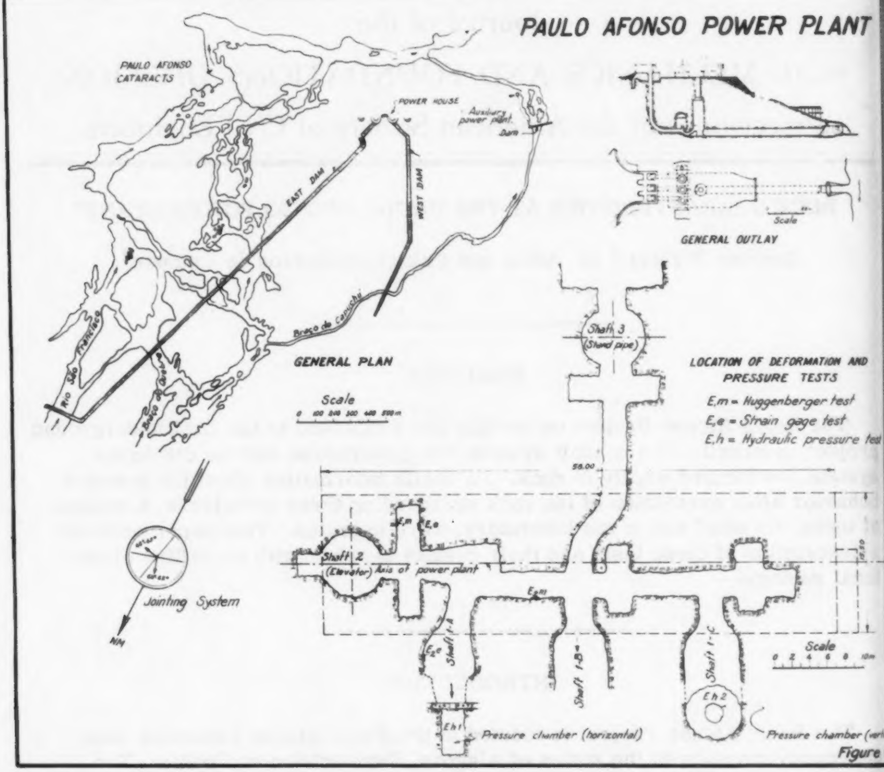


Figure

Outline of Geology

The bedrock of the area consists of a crystalline complex of Precambrian age. The bedrock is covered by a shallow layer of sandy soil that normally is less than 1 meter in thickness. This soil has been formed by the erosion of a Triassic (?) sandstone formation which exists in the form of outliers in this region.

The bedrock formation is a migmatite formed by intrusion of pegmatite and aplite into a biotite schist. Figs. 2 and 3 present views of the rock as seen in the galleries. Due to the tectonic movement that accompanied those intrusions the rock is fractured. Some minor faults and a major fault are responsible for the position of the São Francisco River which has excavated its bed below the cataract along the fault planes where erosion resistance is a minimum.

At the water intake shafts and galleries, two faults occur which have crush zones with thickness of 0.30 m. There is, however, no indication of any geologically recent tectonic movement which might endanger the project or the construction itself. Among the many joints of the rock, the systems N 12-17° E and N 69-72° E are prominent. It was found that by exercising reasonable care, the rock excavation for the galleries, shafts, and power houses could be

Fig. 2. Photograph of the rock in the galleries.

carried out without support. However, because of the general aspect of the rock, it was considered necessary to obtain more precise information on the stress-strain characteristics of the rock in connection with the design of the concrete linings. For securing this information the following tests were carried out.

Deformation and Pressure Tests

To obtain some data on the existing stresses in the rock, deformation tests were carried out. Direct pressure tests were made to determine the modulus of elasticity. Specimens of the two main rock types found in the galleries were also submitted to compression tests in the laboratory to determine the modulus of elasticity.

Deformation Tests

The heterogeneity of the rock formation made information on the variability of stresses in the rock desirable. The stresses are assumed to be proportional to the strains developed upon release of those stresses. The tests were performed on four rock sections. On two sections, a plane surface of about 0.60 m. was prepared and reference marks were cemented in. Seven series of eight holes were drilled around these marks normal to the surface to a depth of 0.15 m. so as to liberate progressively a rock cylinder of 0.50 m. diameter. Before and after each series of holes, the deformation of the rock in different directions was measured with a "Huggenberger" deformometer. Fig. 4 shows one of the test sections and the test results are given in Figs. 5 and 6.

Fig. 3. Photograph of the rock in the galleries.

The vertical deformation by far exceeds that registered in the other directions. Up to 60% of liberation, deformation was less than 0.001 inch.

With additional drilling, the deformation increased rapidly. The maximum deformation occurred in the vertical direction and attained 0.007 inch, whereas the minimum strain occurring in the horizontal direction was little more than 0.001 inch. The negative values seem to indicate that the rock is under tension in some directions so that upon stress release, a small contraction took place.

On two similar but polished surfaces strain gages were applied and relief of the rock stresses was procured in the same way as in the tests already described. Fig. 7 presents an aspect of the test set-up and Figs. 8 and 9 show the general disposition and the test results obtained.

Fig. 8 shows that the rock at this test sections supported, independent of the direction, a tensile stress and an appreciable contraction; negative deformation was registered. The petrographic complexity of the test section may explain that behavior. In Fig. 9, a test section of a more uniform petrographic character is presented and the results are similar to those obtained with the "Huggenberger" test.

Both test procedures indicate that in certain planes, the rock is under substantial stress. The precision of both methods in measuring deformation is approximately the same. However, the Huggenberger deformometer is easier to use than the strain gage.

Pressure Tests

Two hydraulic pressure tests were carried out—one in a horizontal test chamber and the second in a vertical one. The location of these chambers is

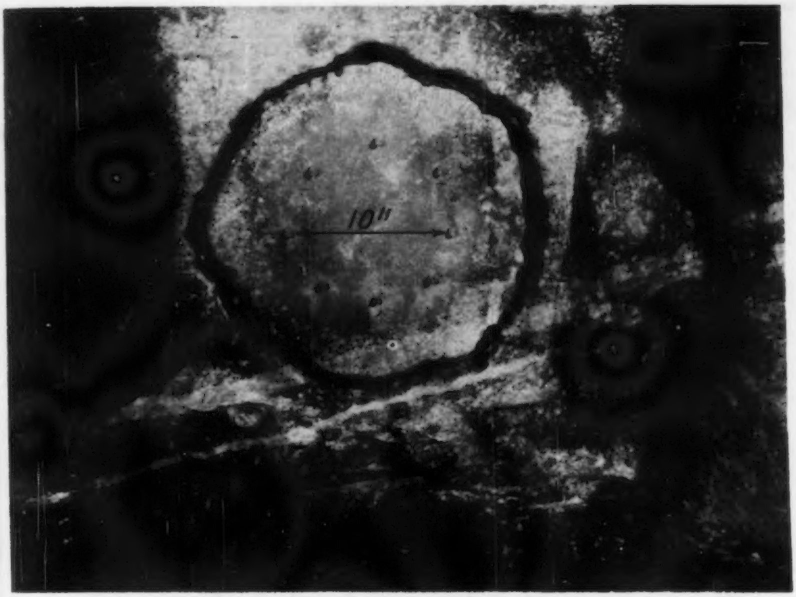
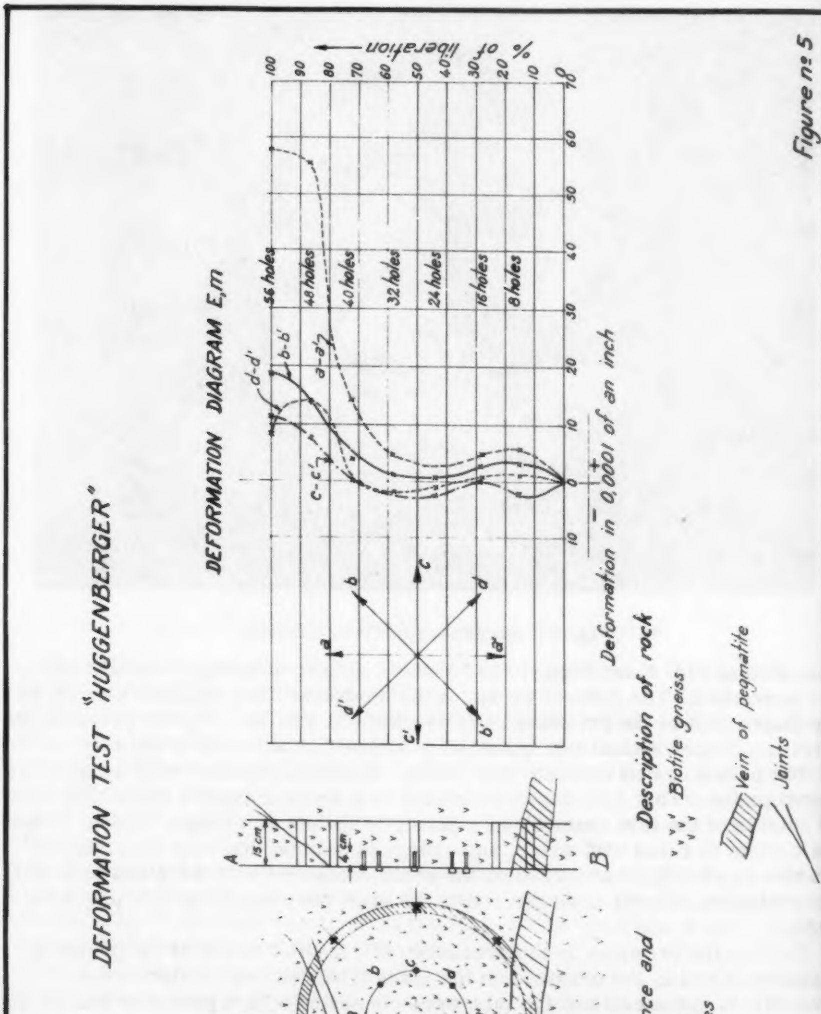


Fig. 4. Huggenberger test section.

indicated in Fig. 1 and Figs. 10 to 13 where details of test devices and outlay are presented. The general aspect of the horizontal test chamber as well as the disposition of the pressure cells is shown in Fig. 10. Fig. 11 presents the situation of the vertical test chamber in connection with one of the main shafts.

The pressure cell consists essentially of a brass cylinder with a tightly adjusted piston. This cylinder is connected to a small diameter glass tube located outside of the test chamber on a panel, by means of a copper tubing. When the system is filled with water, any change in volume that may occur in the pressure cell will be observed on the scale connected with the glass tube and the variances of level are referred to the level corresponding to a pressure zero.

To keep the pressure in the pressure cell, located inside of the pressure chamber, equal to the pressure in the glass tube, located outside of the chamber, both the cell and the tube were connected with a pressure stabilizing chamber. This in turn was connected with a cylinder of compressed air to compensate for any leakage of that air along the system. The inside diameter of the brass cylinder was 75 mm and that of the glass tube was 11,5 mm. Therefore, the linear deformation of the piston in the pressure cell, corresponding to a certain pressure, will be equal to the difference in level registered at the glass tube with reference to the level of pressure zero, divided by 42,5 (ratio of the areas of the section of the brass cylinder and the glass tube). Thus, by pumping water into the pressure chamber readings were taken at the glass tubes, corresponding each one to a certain pressure cell, and the corresponding linear deformation were calculated. The test procedure was identical in the horizontal and in the vertical test chamber.


DEFORMATION TEST "HUGGENBERGER"
DEFORMATION DIAGRAM E,m
% of liberation
36 holes
48 holes
40 holes
32 holes
24 holes
16 holes
8 holes
Deformation in 0,0001 of an inch
Disposition of reference and perforation points
∘ Reference points
x First group of holes
Description of rock
Biotite gneiss
Vein of pegmatite
Joints
50 cm
2,54 cm
15 cm
4 cm
Figure nº 5

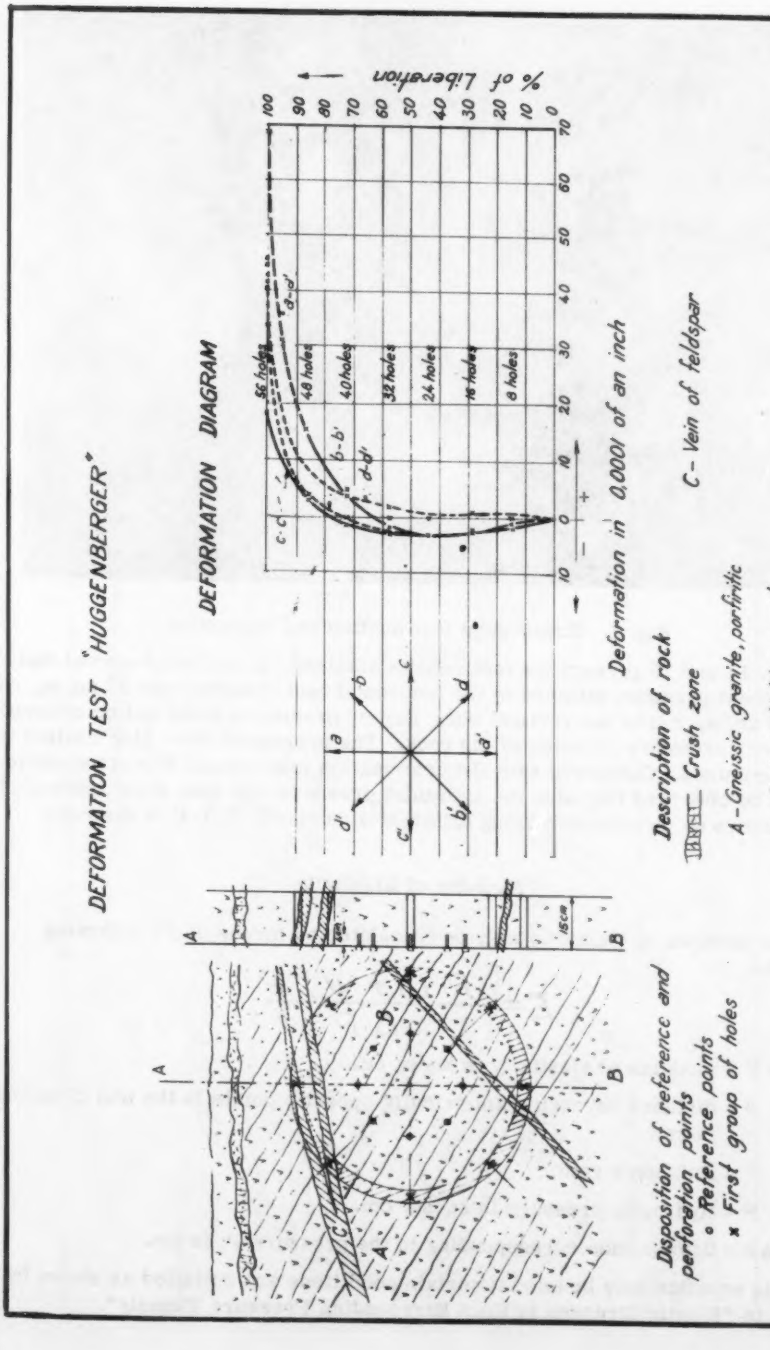

DEFORMATION TEST "HUGGENBERGER"
DEFORMATION DIAGRAM
% of Liberation
56 holes
48 holes
40 holes
32 holes
24 holes
16 holes
8 holes
a-a'
b-b'
c-c'
d-d'
Deformation in 0,0001 of an inch
Disposition of reference and perforation points
∘ Reference points
× First group of holes
Description of rock
Crush zone
A - Gneissic granite, porfiritic
B - Gneissic biotite granite
C - Vein of feldspar
15cm

Figure nº 6

Fig. 7. Strain-gage test section and equipment.

Fig. 14 and 15 present the test results obtained. It will be observed that the highest pressure attained in the horizontal test chamber was 16 kg/sq. cm and 12 kg/sq. cm in the vertical one. Higher pressures could not be obtained even with extensive grouting of the rock. The pressures were also limited by the pump used. Compared with the deformation test results described above, it will be observed that also the hydraulic pressure test data show appreciable differences on deformation along different direction (1-2-3-4) of the rock.

Modulus of Elasticity

The modulus of elasticity may be calculated by means of the following equation.

$$E = \frac{\rho\,(1 + \nu)\,P}{\Delta\rho}$$

where E = modulus of elasticity in kg/sq cm.

ρ = distance between diametrically opposite points in the test chamber in cm.

ν = Poisson's ratio

P = Hydraulic pressure in kg/sq. cm.

$\Delta\rho$ = deformation corresponding to the pressure P, in cm.

This equation may be used if certain conditions are satisfied as shown by Dunn, in "Elastic Stresses in Rock Surrounding Pressure Tunnels".

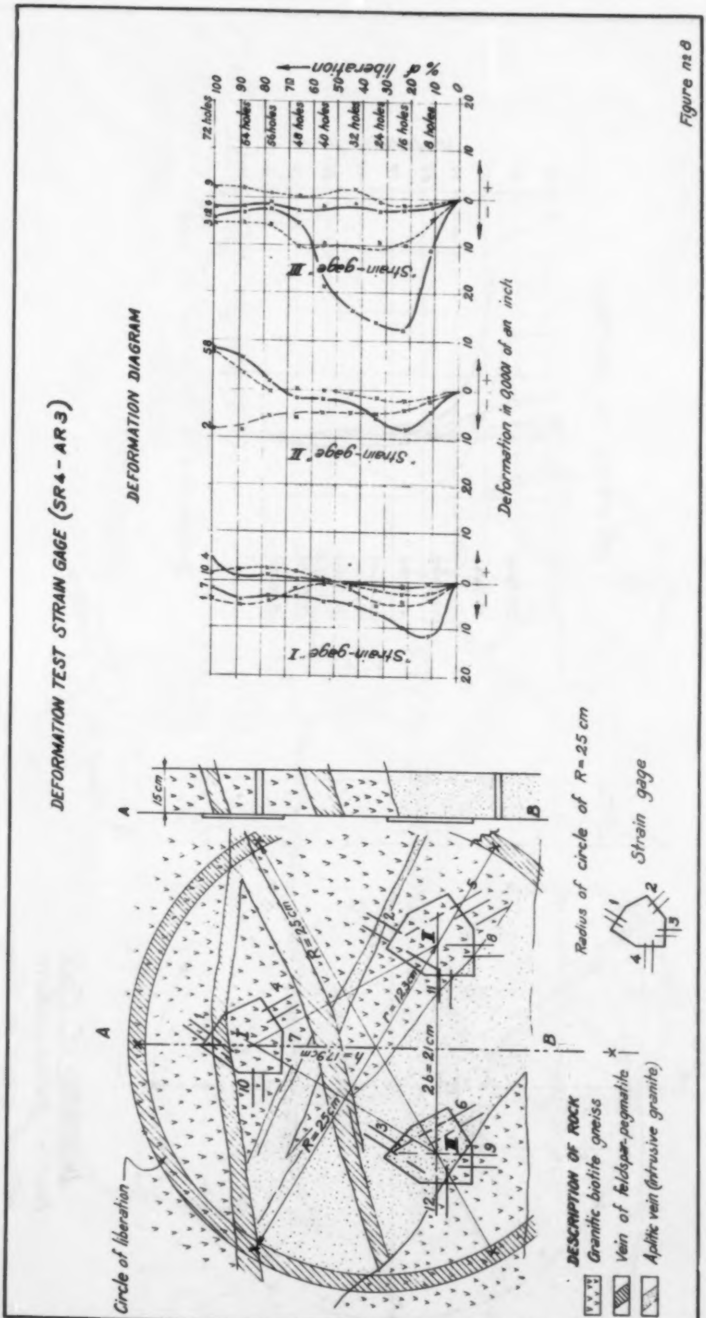

DEFORMATION TEST STRAIN GAGE (SR4 - AR3)
DEFORMATION DIAGRAM
"Strain-gage" I
"Strain-gage" II
"Strain-gage" III
% of liberation
Deformation in 0.0001 of an inch
Circle of liberation
R=25 cm
2b=21 cm
15 cm
DESCRIPTION OF ROCK
Granitic biotite gneiss
Vein of feldspar-pegmatite
Aplitic vein (intrusive granite)
Radius of circle of R=25 cm
Strain gage

Figure no 8

DEFORMATION TEST "STRAIN GAGE" (SR_4-AR_3)

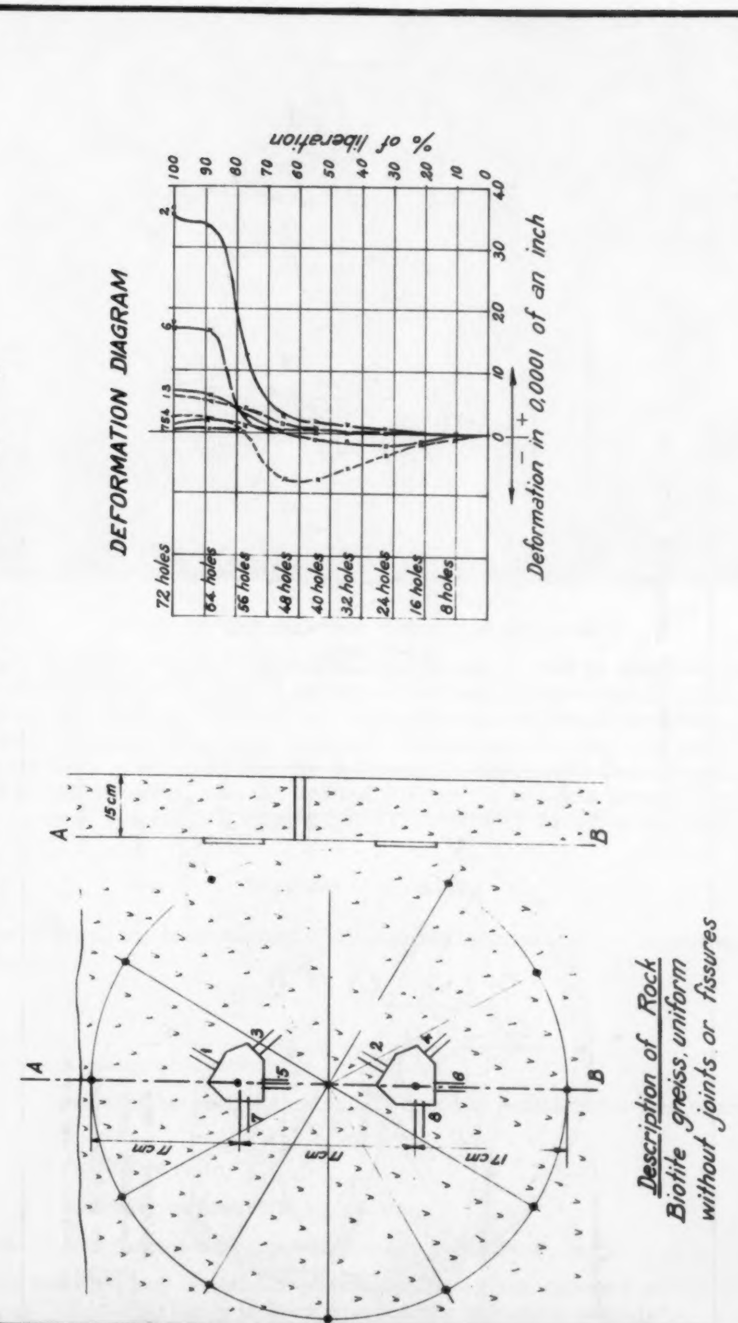


Figure nº 9

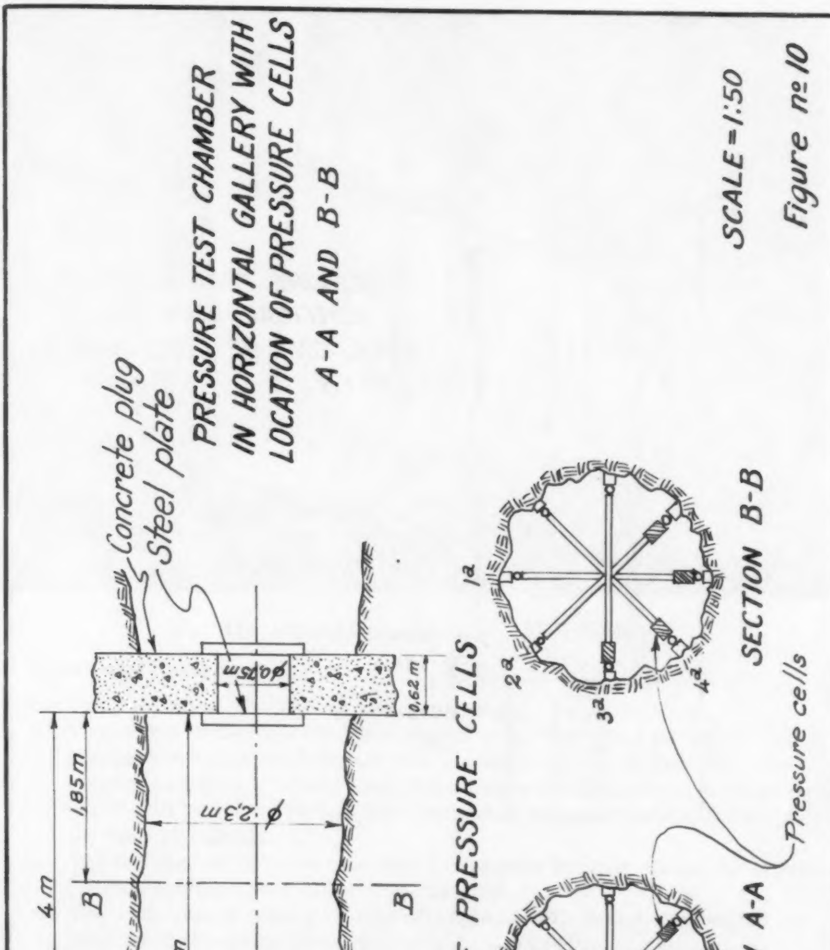

PRESSURE TEST CHAMBER IN HORIZONTAL GALLERY WITH LOCATION OF PRESSURE CELLS A-A AND B-B
Concrete plug
Steel plate
4 m
1,85 m
5,2 m
ø 2,3 m
ø 0,95 m
0,62 m
A
A
B
B
DISPOSITION OF PRESSURE CELLS
1
2
3
4
1a
2a
3a
4a
SECTION A-A
SECTION B-B
Pressure cells
SCALE = 1:50

Figure nº 10

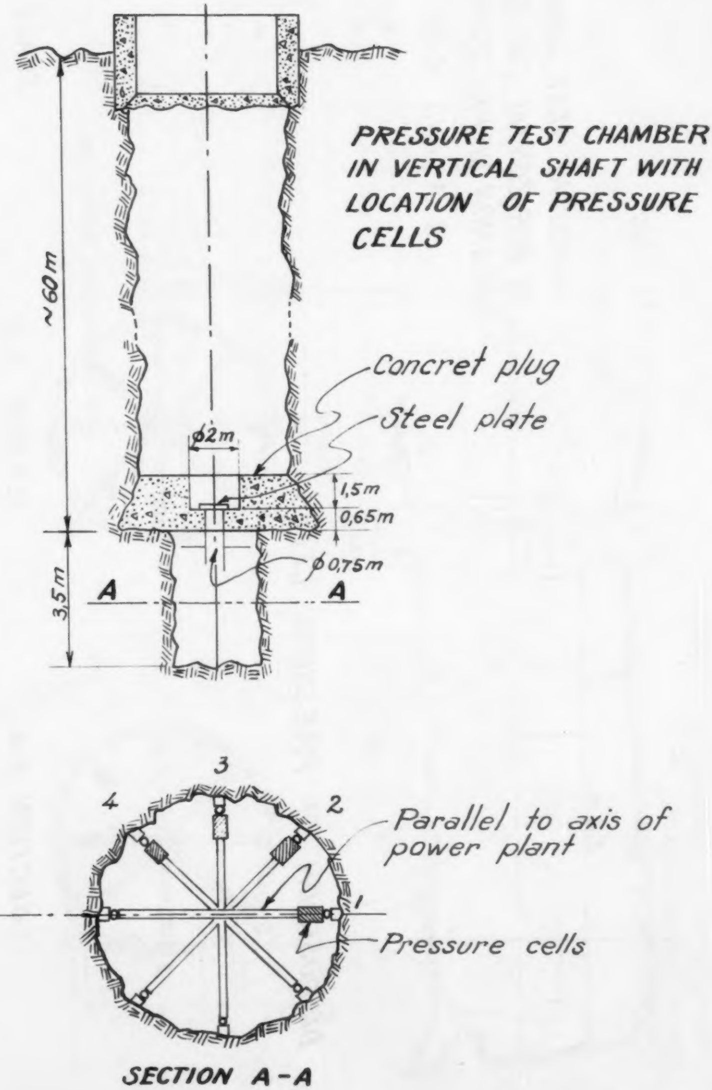

PRESSURE TEST CHAMBER
IN VERTICAL SHAFT WITH
LOCATION OF PRESSURE
CELLS
~60m
Concret plug
Steel plate
ϕ2m
1,5m
0,65m
ϕ0,75m
3,5m
A
A
3
4
2
1
Parallel to axis of
power plant
Pressure cells
SECTION A-A

Figure nº 11

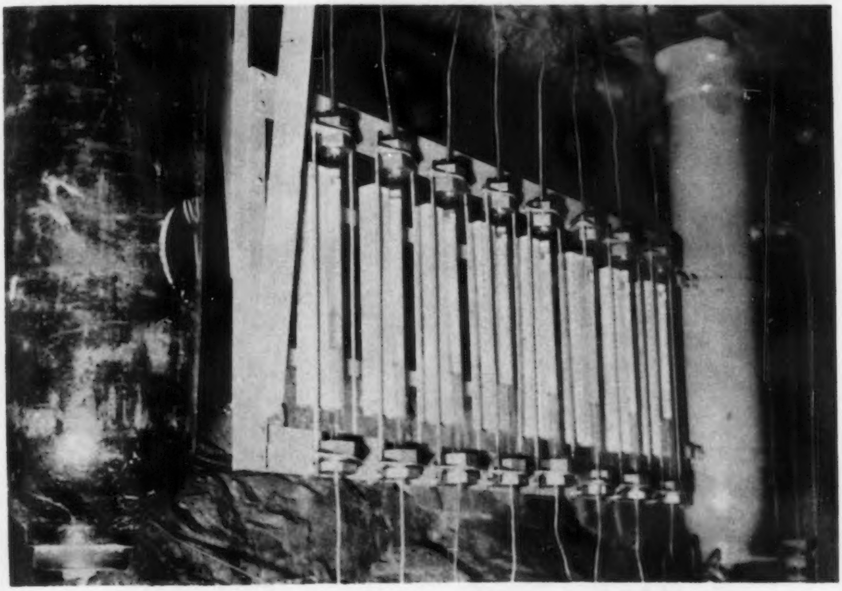


Fig. 12. Photograph of measurement device.

These conditions are:

1. The rock should be homogeneous, isotropic and perfectly elastic.
2. The length of the test chamber should be greater than its diameter. It has been verified analytically that the equation can be applied if the length is equal or greater than three times the diameter. No important error will be committed in this case when the modulus of elasticity of the rock is calculated.
3. The distance of the test chamber to the rock surface should be at least 5 times the diameter of the test chamber.
4. The rock should have a tensile strength able to resist the tension appearing at the chamber periphery.

These conditions, especially the first, are rarely satisfied. In Paulo Afonso, only conditions 2, 3, and 4 were in accordance with the stated requirements so that only approximately correct values were obtained.

However, for the sake of experience and in order to obtain a value, the modulus of elasticity was calculated for every test for different directions. A Poisson's ratio of 0.25 was assumed. The rock deformation considered for the calculation of the elastic modulus was that corresponding to a pressure of 10 kg/sq. cm as indicated in Figs. 14 and 15. The values obtained are presented in Table 1.

The E values vary considerably for different directions, the lowest being around 150 x 103 kg/sq. cm and the highest about twice the value obtained in the vertical test chamber. This variance can be explained by the variance of the petrologic character of the rock in which structure, texture and mineralogical composition are quite variable.

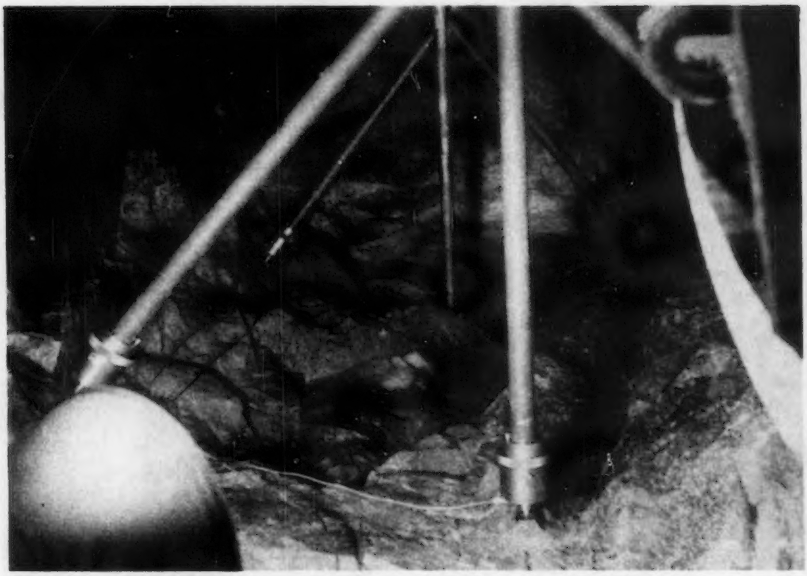


Fig. 13. Pressure cell disposition in the test chamber.

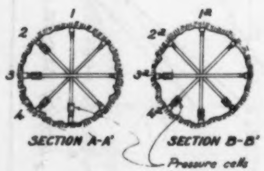
To compare the above results with values from the laboratory, two blocks of different petrological type, selected as representative of the extreme types of rock that comprise the local migmatite, were cut into four smaller cubic blocks. The cubes were tested in compression. Some were tested parallel to the a axis, others parallel to the b axis, and some parallel to the c axis (see Fig. 16 and 17). The deformation was measured along the axis parallel to the direction of load application. The results are presented in the tables of Figs. 16 and 17.

CONCLUSIONS

1. The deformation tests carried out have shown that the stresses in the rock varies to a great extent with the direction; in general, strains are maximum along the vertical and minimum along the horizontal direction.
2. The results of the deformation test indicate the variance of residual stresses in the rock along different directions. The "Huggenberger" test, having a similar, somewhat smaller precision than the "Strain gage" test, presents the advantage of easier execution and may be carried out without interference with the work in the tunnel. For that reason this test may be preferred to the "strain gage" test.
3. The water pressure test results have shown a similar variance of deformation with different rock directions as have been observed by the deformation tests. The modulus of elasticity varies in accordance with these directions. The values obtained are rather low. This may be attributed to the geologic character and the fissures in the rock.

TEST CHAMBER OF GALLERY 1-A

DIAGRAMS OF TEST RESULTS SHOWING THE VARIANCE OF DEFORMATION WITH THE PRESSURES APPLIED



DEFORMATION IN mm ALONG THE DIRECTIONS

PRESSURE kg/cm²	1			2			3			4		
	A	B	C	A	B	C	A	B	C	A	B	C
5	0,09	0,09	0,04	0,11	0,11	0,03	0,1	0,1	0,02	0,04	0,04	0
9	0,14	0,18	0,05	0,11	0,14	0,05	0,1	0,12	0,1	0,1	0,1	0,06
12	0,19	0,24	-	0,11	0,16	-	0,11	0,81	-	0,15	0,21	-
16	0,32	0,39	-	0,15	0,2	-	0,13	0,23	-	0,17	0,23	-

PRESSURE kg/cm²	1ª			2ª			3ª			4ª		
	A	B	C	A	B	C	A	B	C	A	B	C
5	0,07	0,07	-	0,04	0,04	-	0,07	0,07	0,05	0,04	0,04	0,02
9	0,17	-	-	0,1	-	-	0,07	0,12	0,07	0,08	0,1	0,05
12	0,2	-	-	0,09	-	-	0,08	0,15	-	0,09	0,14	-
16	0,27	-	-	0,18	-	-	0,13	0,2	-	0,16	0,21	-

COLUMN A: Parcial deformation due to the application of pressure, independent of residual deformation

COLUMN B: Total deformation since the beginning of the test, taking in account residual deformation

COLUMN C: Residual deformation after release of pressure, deformations in mm

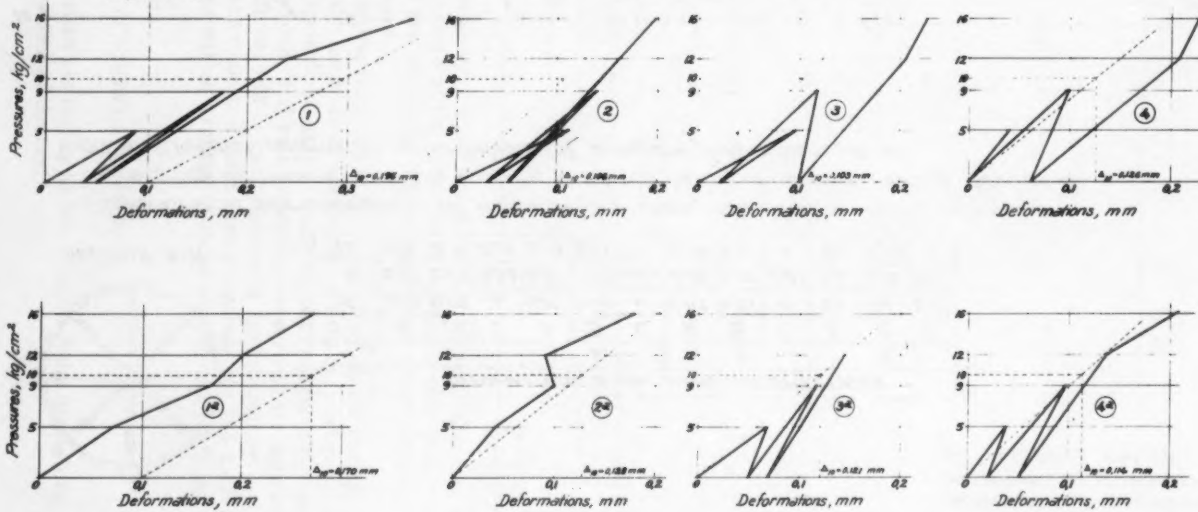


Figure nº 14

TEST CHAMBER OF GALLERY I-A

DIAGRAMS OF TEST RESULTS SHOWING THE VARIANCE OF DEFORMATION WITH THE PRESSURES APPLIED

SECTION A-A

PRESSURE kg/cm²	DEFORMATIONS IN mm ALONG THE DIRECTIONS											
	1			2			3			4		
	A	B	C	A	B	C	A	B	C	A	B	C
5	0,02	0,02	0	0,02	0,02	0	0,07	0,07	0	0,1	0,1	0,02
7	0,07	0,07	0,02	0,05	0,05	0,04	0,08	0,08	0,03	0,1	0,12	0,02
12	0,14	0,16	0,08	0,09	0,13	-	0,08	0,11	-	0,2	0,22	-

COLUMN A: Parcial deformation due to the application of pressure, independent of residual deformation

COLUMN B: Total deformation since the beginning of the test, taking in account residual deformation

COLUMN C: Residual deformation after release of pressure, deformations in mm

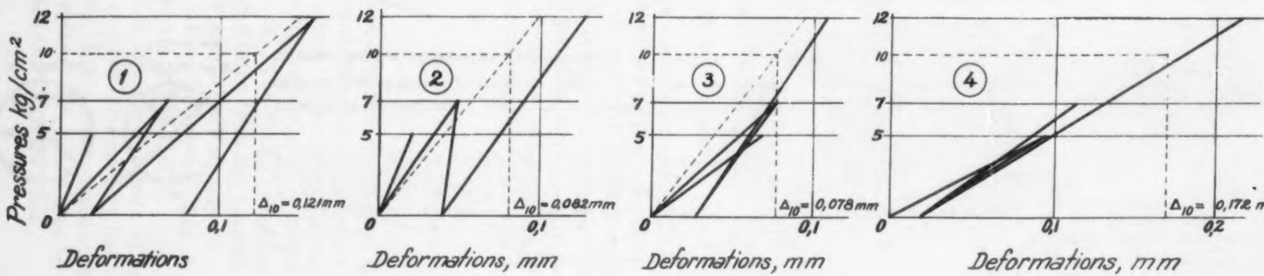


Figure nº 15

DETERMINATION OF THE MODULUS OF ELASTICITY

Block	Rock type	Test esp.	S in cm²	$\varepsilon \cdot 10^5$ test	σ ton/cm²	E ton/cm²	σ rupt ton	Graphic representation of the variance of "E"
1	Aplitic Granite	1a	98,30	41,3	20	495	-	
	"	1b	96,10	27,2	20	770	-	
	"	1c	100,50	74,6	10	103	55,5	
2	"	2a	100,80	60,0	20	330	-	
	"	2b	101,90	50,4	20	394	135,5	
	"	2c	102,10	55,4	10	177	-	
3	"	3a	100 80	47,2	20	422	-	
	"	3b	102,00	31,6	20	620	155,5	
	"	3c	102,60	48,2	10	202	-	
4	"	4a	101,10	51,0	20	388	156,0	
	"	4b	100,20	44.9	20	445	-	
	"	4c	97,90	40,2	10	255	-	
A, B, C	"	A	100,20	49.9	-	407	1240 kg/cm²	Calculated average test values
	"	B	100,00	38,0	-	557		
	"	C	100,80	54,8	-	184		

ton/cm² = kg/sq.cm x 10^3

Figure 16

4. The most erratic results were obtained in the laboratory. However considering the average values of the modulus of elasticity obtained, it will be observed that they are about two times higher than those obtained by the water pressure tests.
5. The results obtained in the field and in the laboratory do not permit a direct correlation of the modulus of elasticity. Tests carried out only in the laboratory, on specific samples, which do not represent the geologic character of the rock, will be misguiding because of the unknown correlation of the tests results in the field and in the laboratory.
6. The modulus of elasticity adopted for design purposes was 200 x 10^3 kg/sq cm which represents an average of those obtained in the test chamber.

ACKNOWLEDGMENT

The writer wishes to express his thanks to Mr. Morris Bohn for editing assistance and the careful review of the text.

TABLE NO. 1

MODULUS OF ELASTICITY IN ACCORDANCE WITH THE PRESSURE

TEST RESULTS AND THE EQUATION $E = \frac{\rho\,(1+\nu)\,p}{\Delta\rho}$

Test gallery	section n:	ρ of cross section cm	p pressure kg/cm²	$\Delta\rho$ deformation cm	ν	E kg/cm²	E average value kg/cm²
horizontal gallery	1	225	10	0,0195	0,25	144.000	
	2	225	10	0,0106	0,25	265.000	
	3	225	10	0,0103	0,25	277.000	
	4	225	10	0,0126	0,25	224.000	227.000
	1a	225	10	0,0170	0,25	165.000	
	2a	225	10	0,0128	0,25	220.000	
	3a	225	10	0,0121	0,25	234.000	
	4a	225	10	0,0114	0,25	246.000	216.000
vertical (shaft)	1	225	10	0,0111	0,25	253.000	
	2	225	10	0,0082	0,25	314.000	
	3	225	10	0,0078	0,25	331.000	
	4	225	10	0,0172	0,25	150.000	262.000

DETERMINATION OF THE MODULUS OF ELASTICITY

Block	Rock type	Test esp.	S in cm^2	$\varepsilon \cdot 10^3$ test	σ ton	E ton/cm^2	σ rupt. ton	Graphic representation of the variance of "E"
1	Melanocratic gneiss	5a	104,70	38,2	20	554	-	
	"	5b	103,60	27,8	20	695	145	a c b
	"	5c	106,20	34,2	20	550	-	
2	"	6a	102,40	24,6	20	795	112	
	"	6b	103,50	23,[illegible]	20	812	-	a c b
	"	6c	103,50	39,0	20	495	-	
3	"	7a	97,60	28,0	20	731	-	
	"	7b	101,20	10,5	20	1883 (?)	-	a c b
	"	7c	97,60	34,0	20	604	98	
4	"	8a	103,00	34,2	20	568	80	
	"	8b	The test especimen collapsed at the beginning of the test					a c b
	"	8c						
A C B	"	A	101,90	31,2	20	662	1070 kg/cm^2	Calculated average test values
	"	B	102,80	20,7	20	1099		A C B
	"	C	102,40	35,7	20	529		

ton/cm^2 = kg/sq cm × 10^3

Figure 17

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SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

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Note: Paper 2140 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 4, August, 1959.

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THE STRUCTURE OF COMPACTED CLAY[a]

Closure by T. William Lambe

T. WILLIAM LAMBE,[1] F. ASCE.—The discussers have contributed to the value of the paper by emphasizing certain parts, by pointing out parts that need clarification, and by noting parts that are controversial. The discussers substantially agree with the author that the physicochemical approach to soil behavior offers a most potent tool to the soil engineer. Space in the Journal and knowledge of the author are both insufficient to permit a complete and satisfactory answer to be given to all questions raised in the discussions (to say nothing of those raised by engineers in letter and by word.) The author will, however, attempt to answer most of the questions.

Dr. Trollope is correct in pointing out that most of the principles of colloidal chemistry were developed for the dilute sols and, for the case at hand, dilute suspensions of clay in water. The author shares Dr. Trollope's concern in using these principles for soil-water systems in which the soil phase is generally larger than the water phase. This point is discussed in the paper, starting on page 1654-10; also discussed are other important reasons why the theories of colloidal chemistry based on idealized conditions are of limited quantitative value to the soil engineer.

The author agrees with several of the discussers that certain of the terms used in the paper are not accurately descriptive. The author attempted to employ commonly used terms rather than coining new ones. As the first sentence of the synopsis of the paper states, structure is a term referring both to the arrangement of particles and the forces acting between them. As pointed out in the second paper (1655) the electrical forces acting between particles and the externally applied forces are of necessity in equilibrium. Thus changes in the externally applied forces must result in changes in the electrical forces between particles, i.e., one of the components of structure. At the present state of knowledge the author finds it difficult to be much more precise in the numerical definition of such terms as structure, dispersion and flocculation. It is hoped that further knowledge will permit this improvement, and a selection of better terms.

The author must disagree with several points made by Professor Warkentin and Mr. Yong. The discussers state that the double layer theory predicts only a small influence of surface charge density on the thickness of the diffuse ion-layer. In arriving at this conclusion the discussers apparently assumed (incorrectly) that the concentration of ions at the midplane between colloids was independent of the surface charge.

a. Proc. Paper 1654, May, 1958, by T. William Lambe.

1. Director, Soil Stabilization Lab., Prof. of Soil Mechanics, Massachusetts Inst. of Technology, Cambridge, Mass.

Professor Warkentin and Mr. Yong feel that there is no need for an electrical attraction to exist for soil particles to flocculate in a clay-water suspension. The author cannot accept this view. In fact, are not the discussers implying a net force of attraction when they state that ". . . the particles approach each other more closely and act in unison . . ."?

Professor Warkentin and Mr. Yong are correct in suggesting that cementing substances may help hold the particles together in natural clays. The author has presented elsewhere (Lambe and Martin, 1956) experimental evidence on the cementation of fines in natural clays by iron oxides, carbonates, and organic matter.

Professor Leonards takes exception to the relatively minor role the author gives to any unusual structure of soil water. The paper uses the term "double-layer water" in a general sense to cover all water acted on by significant attractive forces from the soil. The innermost part, and most strongly held part, of the double layer water is termed "adsorbed water." See Fig. 7. The molecules of adsorbed water are undoubtedly oriented, those in the water farther from the soil to a less degree.

The writer feels that adsorbed water has physical and chemical properties quite unlike free water as would exist in a beaker of water. It, for example, certainly appears to be far more viscous than free water. This adsorbed water plays an important role in such phenomena as secondary compression and creep.

The double layer water beyond the adsorbed layer serves mainly to transmit electrical forces. The author explained soil behavior in terms of adsorbed water, electrical forces, and outer double-layer water; the latter water having a structure and properties approximating those of free water. The author sees no need to postulate any "ice-type" structure in this water; he does not feel there is conclusive proof of its existence.

Professor Leonards has done a commendable job in citing the literature to show that soil water indeed behaves quite differently from free water. The available experimental data have been used both by those who show that water plus ions as a single component in a clay behaves differently from free water and by those who have attempted to show that the unusual behavior can be explained by the ions with only a relatively small contribution from any unusual structure of the water phase itself. For example, Michaels and Lin (1954) interpret their experiments on the permeability of kaolinite as proof that the thickness of immobilized water on the kaolinite was very thin, of the order of 10Å. Professor Werner E. Schmid (1957) used the same data obtained by Michaels and Lin to help support his stationary boundary-layer principle, which attributes the unusual permeability properties of clays to the existence of a thick adsorbed water layer.

On the basis of the theoretical principles presented in the paper, the author was led to the prediction that a decrease in temperature would reduce the shear strength of a clay. Laboratory experiments (see Fig. 3) have shown that heating a suspension tends to cause the particles to stick together and flocculate. Further, as discussed in the second paper (1655) on page 1655-11 there are experimental data to show that cooling a sample will cause it to expand. These two facts are interpreted as an indication that a reduction in temperature causes an increase in electrical repulsion between adjacent particles.

The author has obtained experimental data on two different clays to show that cooling a sample, while maintaining pore pressures, zero thus permitting

the sample to swell, will cause a reduction in the undrained shear strength of the soil. The Rosenqvist data, cited by Professor Leonards, was obtained on unconfined compression samples (personal communication from Dr. Bjerrum). Cooling an unconfined sample of clay could result in the development of higher pore water tensions to resist the expansion of the clay. In other words, the externally applied effective stress has been increased by the cooling; this increase in effective stress would naturally tend to offset the increase in electrical repulsion.

The M.I.T. Soils Laboratory now has in progress a test program which, among other things, will obtain shear strength of clay at various temperatures. This program has not advanced far enough to permit a definitive statement on the effect of temperature on shear strength. In light of the data presently available, the author still sticks to his prediction that (other things being equal) a reduction in temperature will cause a decrease in shear strength if the effective stress remains constant. This prediction refers to a variation of temperature at the time of test. In other words, consider two samples which are identical in all respects; cool one and heat one while maintaining the pore pressures equal, e.g., zero; test the two samples; the hot one should have a higher undrained strength than the cool one. There is some evidence to show that low temperature consolidation may produce more strength than high temperature consolidation under the same pressure increment. Additional data may necessitate a more careful definition of shear strength or indicate limitations or exceptions to the concept.

Professor Krynine has contributed helpful comments on several parts of the paper. The author heartily agrees with Professor Krynine that the nature and extent of contact between adjacent clay particles needs additional study.

Like Professor Krynine, the author would expect that, in general, marine clays would have a less dense structure than fresh water clays. Figs. 4 and 5 were not intended to suggest otherwise. A structure such as indicated in Fig. 4a (salt flocculation) is probably typical of the structure of a freshly deposited marine clay. A typical fresh water clay should have a structure somewhere between b (non-salt flocculation) and c (dispersion) or even between a and c. It seems unlikely that any fresh water clay would be deposited in such pure water as to permit an arrangement consisting entirely of edge-to-face flocculation as shown in Fig. 4b and at the far left of Fig. 5.

The author suggested shrinkage upon drying as a crude and easily obtained measure of structure. Mr. Freitag's procedure of measuring shrinkage to a given relative humidity and then measuring expansion under controlled conditions certainly gives more information than a simple shrinkage measurement. Mr. Freitag's method is, unfortunately, very time consuming.

In conclusion the author would like to express his appreciation to those who prepared and submitted discussions to his paper.

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THE ENGINEERING BEHAVIOR OF COMPACTED CLAY[a]

Closure by T. William Lambe

T. WILLIAM LAMBE,[1] F. ASCE.—The six discussers have added to the value of the paper with their comments. The author will attempt to answer the questions raised and, to some extent, furnish further information where requested.

Professor Scheer wonders about the differences in horizontal and vertical permeability as a function of structure. As one would suppose, the flocculated structure shows permeability in all directions approximately equal. Upon particle orientation, the permeability of a soil becomes non-isotropic with the permeability being less for flow perpendicular to the long axis of the plates than for flow parallel. In other words, one-dimensional compression in the vertical direction will line the particles horizontally; the ratio of horizontal to vertical permeability will increase.

The author had hoped to use this concept to measure particle orientation in a soil mass by the ratio of vertical to horizontal permeability. While the concept is apparently sound, the differences in permeability with direction are usually relatively small (typical ratio values of 2 or 3). This fact (along with others) indicates that a high degree of particle alignment can occur in zones, with different alignments between zones, i.e., random oriented packets of parallel particles.

Mr. Yong and Professor Warkentin have misinterpreted Fig. 2 of paper 1654. Fig. 2d shows that increasing the temperature term in the theory expands the double layer; however, the increase in temperature affects the dielectric term to such an extent that the influence of the temperature term itself is reversed. In other words, the net influence of a change in temperature and the corresponding change in dielectric term is an expansion of the double layer with a decrease in temperature. This combined effect is shown in Fig. 2e.

That an increase in temperature generally tends to cause flocculation is an experimental fact (see illustration of this fact in Fig. 3d, paper 1654). This fact can be explained solely by the classic double-layer theory (Fig. 2e, paper 1654). There is a component, not covered by the Gouy-Chapman theory, to temperature's role in electrical forces. An increase in temperature increases the rate of force dissipation between dipoles by tending to give a more uniform orientation distribution in the dipoles.

Mr. Turnbull of Australia has raised objections that need examination. He states that the two soils for which permeability data are presented in Figs. 1 and 2 are not "true clays" and ". . . would not contain any appreciable proportions of true clay minerals . . ." Both soils behave as clays and both

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a. Proc. Paper 1655, May, 1958, by T. William Lambe.

1. Director, Soil Stabilization Lab., Prof. of Soil Mechanics, Massachusetts Inst. of Technology, Cambridge, Mass.

contain "true" clay minerals; both, however, contain silt and sand size particles. The Jamaica soil termed by the author as "sandy clay" contain about 10% kaolinite; the Siburua clay contains about 50% of "true" clay minerals—illite, montmorillonite, and kaolinite in equal portions. The permeability of a soil is determined largely by the finer particles rather than the coarser ones. A prepared mixture of 10% montmorillonite clay and 90% sand will have a permeability much closer to that of 100% montmorillonite than to that of 100% sand.

Mr. Turnbull has used the optimum water content as an indication of the behavior of a clay. The unsoundness of this procedure is clearly shown by the following test data (Cornell, 1951).

Clay	Plastic Limit	Plasticity Index	Optimum Water (Proctor Compaction)
Na Montmorillonite	54%	650%	22%
Fe Montmorillonite	75%	220%	29%
Na Illite	53%	63%	25%
Fe Illite	49%	59%	27%
Na Kaolinite	32%	21%	17%
Fe Kaolinite	37%	22%	21%
Attapulgite	150%	130%	66%

These data should suggest caution to Mr. Turnbull in the broad use of optimum water content values.

The compaction tests employed the procedure described in the author's book on testing; tempering was used.

Mr. Turnbull and the author are apparently far apart on their concepts of shear strength. Much of the difference may reside in lack of conformity on the meaning of terms. A complete answer to all of Mr. Turnbull's objections to the paper would have to be preceded by an agreement of definitions. In both Figs. 9 and 10 the stress plotted on the horizontal axis is "intergranular" or "effective" stress which is equal to the combined pressure minus the pore pressure. This definition of stresses is widely accepted. Any capillary pressures would have been properly accounted for.

Mr. Turnbull is correct in stating that clay compacted wet of optimum can have negative pore water pressures. The author stated (page 1654-22) that the pore water tensions dry of optimum would be greater than those for compaction wet of optimum. That this can be true is shown by the following experimental data obtained by Mr. Solorzano, a thesis student of the author:

2% dry of optimum; pore pressure = -0.4 kg/cm^2
Optimum; pore pressure = -0.3 kg/cm^2
2% wet of optimum; pore pressure = -0.1 kg/cm^2.
(atmos. pressure is taken as zero)

In view of Professor Salas' pioneering work on soil structure, his discussion merits study. The author readily agrees that physicochemical theory has not completely solved the fundamental problems of soil behavior. In fact the author started paper 1654 with, "as the reader will see, several facets of structure are not fully understood. The author does not, therefore, present

this paper as the final word, but rather as a stimulant and, hopefully, a guide to further studies on this complex subject."

Professor Salas apparently differs from the author's view that sufficient pressure and time can transform a clay into a system that will not disperse. The author considers a true compaction shale an example of just this phenomenon.

The colloidal theory was presented as a general guide. Page 1654-10 clearly states there are . . . "several important factors neglected in colloidal theories . . ." As noted on page 1654-12 . . . "attempts to use numerical equations based on colloidal theory in soil engineering problems have met with failure . . ." The author, as does Dr. Trollope, doubts that theories developed for particles far apart can be numerically used for the closed spaced particles occurring in the typical natural soil.

Professor Salas has presented an interesting approach to soil structure which is based entirely on energies rather than on energies and forces. The author would feel that the force approach, as used in the paper, is superior, especially for natural clays, since it better allows one to consider forces (such as to edge-to-face linkage) which are ignored in the colloidal theories.

Professor Salas takes issue with the author's view that an increase in electrical repulsion results in a decrease in strength and an increase in attraction in an increase in strength. Professor Salas cites as his proof . . . "the greater the external force, the greater the repulsive forces . . . shear strength will be greater . . ." Increasing the external force increases not only the repulsion component but also the attraction and the interaction components, because the force equilibriums stated on page 1655-16 exist. The concept in the paper is right, but not as clearly presented as it could have been. The discusser's objection does not seem to be real in fact, rather a different interpretation of the same concept.

The author agrees with Professor Salas that the inclusion of a statistical concept with the principles of structure is helpful.

The author strongly endorses Dr. Trollope's view on the importance of contact interaction.

In conclusion the author expresses appreciation to those who prepared discussions to his paper. They have emphasized both the importance and the incomplete state of knowledge on the subject.

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PREDICTING SEEPAGE UNDER DAMS ON MULTI-LAYERED FOUNDATIONS[a]

Closure by Paul H. Shea and Harry E. Whitsett

PAUL H. SHEA,[1] and HARRY E. WHITSETT.[2]—The authors wish to commend Mr. Cedergren for demonstrating the practicability of graphical construction of flow nets for estimating seepage through multi-layered foundations of impervious dams when the permeabilities of the various layers differ greatly. In cases in which the authors' approximate analytical solutions do not apply, such as situations with unusual boundaries or with the uppermost layer of the foundation highly permeable, flow net construction is the only practicable method available. However, the mathematical solutions, whenever applicable, present some compelling advantages.

If a soil is homogeneous but not isotropic, a transformation of scales is needed before Laplace's equation will apply, making possible the use of flow nets in the solution of seepage problems. The transformation can either shrink dimensions in the direction of greatest permeability or stretch dimensions in the direction of least permeability because the flow net is determined by the shape rather than the size of the region. The permeability value used for the isotropic transformed section is so selected that seepage rates are the same in natural and transformed sections.

It is not necessary to use an "average" transformation factor. Consider a blanketed aquifer in which both materials have permeabilities that are different in horizontal and vertical directions. In each region the above considerations apply and the appropriate transformation can be made with a change in vertical dimensions only. This involves no slipping between the regions at their common boundary. Even though different transformation factors may be required on the two sides of the boundary, continuity of head and of the vertical component of velocity at the boundary are preserved. Thus, the transformation is acceptable. These assertions can be justified mathematically (see Appendix A). The extension to cases involving more blankets and aquifers is obvious.

Many problems are susceptible of solution by either the graphical or the mathematical method. In the case of one blanket and one aquifer, Mr. Cedergren's charts and the mathematical solution are both acceptable and are about equally convenient. The construction of a flow net would require much more time and effort. Extension of Mr. Cedergren's charts to cases of more than one blanket and more than one aquifer is not feasible due to the great number of charts required by the many possible permutations of thicknesses

a. Proc. Paper 1727, August, 1958, by Paul H. Shea and Harry E. Whitsett.
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2. Mathematician, Foundations and Materials Branch, U. S. Army Engr. Dist., Jacksonville, Fla.

and permeabilities of the soil strata. The construction of flow nets in these cases may be a relatively simple problem for an expert with wide experience, but it would present a rather formidable challenge to the average soils engineer. Applications of the mathematical solutions require no special skill—only arithmetic and elementary algebra are needed.

Soils are virtually always somewhat stratified and therefore anisotropic. A transformation of scales and permeabilities is required so that Laplace's equation will apply and the graphical construction of a flow net will be practicable. The difficulty here is that the transformation factors are seldom if ever known accurately and are often unknown. Uncertain values of the transformation factors require that even a precise solution of a boundary value problem in the transformed section be regarded as an approximate solution of the practical seepage problem.

If field conditions conform reasonably well to the stated assumptions in the authors' approximate analytical solutions, anisotropic conditions in the foundation should have a relatively minor effect on the accuracy of the seepage computations. It can be demonstrated (see Appendix B) that the transformations required by anisotropy in the case of a flow net solution would have no effect upon the approximate analytical solution of a seepage problem. Thus, the transformation factors need not be known. A field pumping test of the type described in the original paper provides the only really satisfactory method for obtaining the permeabilities required for underseepage computation. Permeability values obtained in that way are those in the desired directions—vertical in the blankets and horizontal in the aquifers.

In the discussion a numerical example was given:

$$L = 275\ \text{ft.}, \quad H = 50\ \text{ft.},$$

$$(k_x)_1 = 4\times10^{-4}\ \text{ft./min.}, \quad (k_z)_1 = 1\times10^{-4}\ \text{ft/min.}, \quad z_1 = 5\ \text{ft.},$$

$$(k_x)_2 = 0.04\ \text{ft./min.}, \quad (k_z)_2 = 0.01\ \text{ft./min.}, \quad z_2 = 50\ \text{ft.}$$

The seepage rate, Q, for a one foot length of the dam, is found to be 0.116 cubic feet per minute (using shape factor from Table I, p. 59). Solution by the approximate analytical method follows:

$$C_1 = \frac{(k_z)_1}{z_1} = \frac{1\times10^{-4}}{5} = 2\times10^{-5}.$$

$$C_2 = (k_x)_2\, z_2 = 0.04\times50 = 2.0 \quad .$$

$$x_r = \sqrt{\frac{C_2}{C_1}} = \sqrt{1\times10^{5}} = 100\sqrt{10} = 316.$$

$$Q = \frac{C_2 H}{L+2x_r} = \frac{2\times50}{275+632} = 0.110 \text{ cubic feet per minute.}$$

The form of the equation for underseepage can be changed to facilitate comparison with Mr. Cedergren's charts.

$$Q = \frac{C_2 H}{L + 2x_r} \qquad \frac{k_1 H}{k_1 H} = \frac{\dfrac{k_2 z_2}{k_1}}{L + 2\sqrt{\dfrac{C_2}{C_1}}}\, k_1 H$$

$$= \frac{\dfrac{k_2}{k_1}}{\dfrac{L}{z_2} + \dfrac{2}{z_2}\sqrt{\dfrac{z_1 z_2 k_2}{k_1}}}\, k_1 H$$

$$Q = \left[\frac{\dfrac{k_2}{k_1}}{\dfrac{1 + \dfrac{z_1}{z_2}}{\dfrac{z}{L}} + 2\sqrt{\dfrac{z_1}{z_2}\,\dfrac{k_2}{k_1}}} \right] k_1 H \quad ,$$

where $z = z_1 + z_2$. The expression in brackets corresponds to Mr. Cedergren's shape factor. Values given by this expression were found to differ from those given by the shape factor charts by less than 15% when $5 < \frac{k_2}{k_1} < 1000$, and by less than 10% when $20 < \frac{k_2}{k_1} < 200$. It is believed that the discrepancy between values given by the two methods when $\frac{k_2}{k_1}$ is large is due almost entirely to the difficulty of construction of the flow nets since accuracy of the approximate analytical method should increase as $\frac{k_2}{k_1}$ increases.

In cases where the assumptions of the authors' approximate solutions are reasonably well met, either flow nets or the mathematical solutions may be used with acceptable accuracy. The mathematical solutions have the advantage of requiring no knowledge of the relative values of permeability in the horizontal and vertical directions, provided that the permeabilities are obtained from pumping tests. A second advantage is that the mathematical solutions can be used by any one with a knowledge of algebra, and special skill in the art of flow net construction is not required. For the average soils engineer, use of the mathematical solutions will save much time in analyzing problems involving more than one aquifer.

APPENDIX A

Proof That Different Transformation Factors Can Be Used for Blanket and Aquifer

Consider two-dimensional flow in an infinite strip so oriented that the x-axis lies within it. Call the regions above and below the x-axis regions 1 and 2. Let each region be homogeneous but not isotropic with respect to permeability, the greatest permeability being either parallel to or perpendicular to the x-axis. The solution of the problem would consist of solutions, P_i (x, y), of the equations

$$(k_x)_i \frac{\partial^2 P_i}{\partial x^2} + (k_y)_i \frac{\partial^2 P_i}{\partial y^2} = 0, \quad (i = 1, 2). \tag{1}$$

Each P_i must satisfy the boundary conditions imposed upon its region. The solutions, P_i, in the two regions must together satisfy conditions of continuity of hydraulic head and of the normal component of velocity at the common boundary of the two regions:

$$P_1(x, 0) = P_2(x, 0), \tag{2}$$

$$(k_y)_1 \frac{\partial}{\partial y} P_1(x, 0) = (k_y)_2 \frac{\partial}{\partial y} P_2(x, 0). \tag{3}$$

If there are no sources or sinks in the regions, then the P_i must be bounded there.

The transformations

$$x' = \frac{\alpha_i}{\sqrt{(k_x)_i}}\, x, \qquad y' = \frac{\alpha_i}{\sqrt{(k_y)_i}}\, y,$$

where the α_i are constants, convert Eqs. (1) into Laplace's equation. When transformed, Eqs. (2) and (3) will have sensible physical meanings only if the transformation of x is the same for both regions. This requires that

$$\alpha_i = \beta \sqrt{(k_x)_i}$$

where β is a constant. Let

$$\zeta_i = \sqrt{\frac{(k_x)_i}{(k_y)_i}}, \quad (\zeta_i > 0).$$

The transformations now become

$$x' = \beta x, \quad y' = \beta \zeta_i y.$$

The transformed regions must be homogeneous and isotropic since Laplace's equation applies to them. If rates of flow are to be determined in the transformed section, then the isotropic permeabilities there, k_i', are required. Flow through corresponding parts of the natural and transformed sections

must be equal. For simplicity, consider flow through a vertical element of length dy located at a point (a,b). The subscript i may be suppressed.

$$dq = v_x(a,b)\,dy = -k_x \frac{\partial}{\partial x} P(a,b)\,dy.$$

$$dq' = v_x'(a',b')\,dy' = -k' \frac{\partial}{\partial x'} P\left(\frac{a'}{\beta}, \frac{b'}{\beta\zeta}\right) dy'.$$

$$= -k'\left[\frac{\partial}{\partial x} P(a,b) \frac{dx}{dx'}\right] \frac{dy'}{dy}\,dy = -\zeta k' \frac{\partial}{\partial x} P(a,b)\,dy.$$

$$dq' = dq.$$

$$-\zeta k' \frac{\partial}{\partial x} P(a,b)\,dy = -k_x \frac{\partial}{\partial x} P(a,b)\,dy.$$

$$k' = \frac{k_x}{\zeta} = \sqrt{k_x k_y}.$$

Thus $$k_i' = \sqrt{(k_x)_i (k_y)_i}.$$

Since β serves only to determine the size of the transformed regions, we may set $\beta = 1$. The transformations alter Eq. (2) only by placing an accent on the x's.

Eq. (3) transforms as follows:

$$(k_y)_1 \frac{\partial}{\partial y} P_1(x,0) = (k_y)_2 \frac{\partial}{\partial y} P_2(x,0). \qquad (3)$$

$$(k_y)_1 \frac{\partial}{\partial y'} P_1(x',0) \frac{dy'}{dy} = (k_y)_2 \frac{\partial}{\partial y'} P_2(x',0) \frac{dy'}{dy}.$$

$$\zeta_1 (k_y)_1 \frac{\partial}{\partial y'} P_1(x',0) = \zeta_2 (k_y)_2 \frac{\partial}{\partial y'} P_2(x',0).$$

But

$$\zeta_i (k_y)_i = \sqrt{(k_x)_i (k_y)_i} = k_i', \quad \text{so}$$

$$k_1' \frac{\partial}{\partial y'} P_1(x',0) = k_2' \frac{\partial}{\partial y'} P_2(x',0).$$

Thus, the stated problem is reduced to the corresponding problem for isotropic regions by the transformations:

$$x' = x, \quad y' = \zeta_i y, \quad k_i' = \sqrt{(k_x)_i (k_y)_i}, \quad (i = 1, 2),$$

where $\zeta_i = \sqrt{\dfrac{(k_x)_i}{(k_y)_i}}, \quad (\zeta_i > 0).$

APPENDIX B

Proof that Approximate Analytical Solution is Unaffected by Transformation

In each case, the underseepage, Q, per unit length of dam is a function of the head differential, H, the base width of the dam, L, and the transmissibilities of the soil strata, C_i. The transformations required for a solution of the problem by graphical construction of a flow net do not affect H, and can be so selected that they do not affect horizontal dimensions, which include L.

Let

$$\zeta_i = \sqrt{\frac{(k_x)_i}{(k_z)_i}}, \quad (\zeta_i > 0).$$

Apply the transformations

$$x' = x, \quad z_i' = \zeta_i z_i, \quad k_i' = \sqrt{(k_x)_i \, (k_z)_i}.$$

When i is odd,

$$k_i' = \sqrt{(k_x)_i} \sqrt{(k_z)_i} = \zeta_i (k_z)_i, \quad \text{and}$$

$$C_i' = \frac{k_i'}{z_i'} = \frac{\zeta_i (k_z)_i}{\zeta_i \, z_i} = \frac{(k_z)_i}{z_i} = C_i.$$

When i is even,

$$k_i' = \sqrt{(k_x)_i} \sqrt{(k_z)_i} = \frac{(k_x)_i}{\zeta_i}, \quad \text{and}$$

$$C_i' = k_i' z_i' = \frac{(k_x)_i}{\zeta_i} \zeta_i z_i = (k_x)_i \, z_i = C_i.$$

Thus, the transformations leave all the arguments of the function Q unaltered and cannot affect Q.

DESIGN AND PERFORMANCE OF VERMILION DAM, CALIFORNIA[a]

Closure by K. Terzaghi and T. M. Leps

K. TERZAGHI,[1] Hon. M. ASCE, and T. M. LEPS,[2] F. ASCE.—The authors were particularly pleased to note that their paper stimulated six engineers, who were closely associated with the design and measurement of performance of Vermilion Dam, to draw upon their personal experiences with the project in the preparation of discussions of the paper. Each discussion supplements the paper with valuable thoughts and experiences.

Mr. Dickinson outlines the design and successful performance of the outlet conduit through the dam. This reinforced concrete, articulated joint conduit was founded on heterogeneous glacial debris having a depth of over 200 feet. Mr. Dickinson mentions that a maximum settlement of 0.27 foot has been measured in the conduit. This occurred under a maximum fill height of 145 feet. The reason that settlement was so minor undoubtedly lies in the finding of our engineering geologists that the site had been glacially pre-loaded by at least several hundred feet of glacial ice. The conduit was actually designed to take much more than the limited deformation expectable under the "pre-load" concept; and actual experience proved that it could safely have been designed to be less articulated.

Mr. Kiely emphasizes the degree of flexibility in design concepts maintained throughout the construction period by the engineers and the owner. This, indeed, is perhaps the key theme of the authors' paper. Mr. Spencer supplements this theme with special reference to the functions of the Consulting Board.

Mr. West presents a particularly valuable analysis of the underseepage experienced at the dam and isolates the significant effects of varying meteorological conditions from the more obvious consequences of changes in reservoir level.

Dr. Birman, who spent many energetic months in detailed geological examination of the damsite and reservoir area, sheds further light, from a glacial geologist's viewpoint, on why the damsite was considered and is adequate for supporting the dam, as it was built.

Mr. Laverty presents the program which was adopted to check on the performance of the dam, and pays tribute to those who have so diligently and painstakingly made the required observations.

Thus, the discussions which have been presented can be viewed as most valuable supplements to the story of Vermilion Dam. The authors wish to

a. Proc. Paper 1728, August, 1958, by K. Terzaghi and T. M. Leps.

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2. Mgr. of Organization & Procedures (formerly Chf. Civ. Engr.), Southern Calif. Edison Co., Los Angeles, Calif.

express their great appreciation to the individual discussors. They also are gratified to learn that the paper, with its discussions, is currently being used by some professors of soil mechanics, such as Dr. R. V. Whitman of M.I.T., as an advanced level case study in their courses of instruction.

EFFECTS OF GROUND ON DESTRUCTIVENESS OF LARGE EARTHQUAKES[a]

Closure by C. Martin Duke

C. MARTIN DUKE,[1] F. ASCE.—The discussions by Mr. Eremin and Mr. Ambraseys have treated, respectively, the design implications of the paper and the response of soil to seismic stress. The author appreciates these contributions.

The empirical data available to date clearly point to the probability that the character of the structure and its foundation, together with the character of the ground, determines the destructiveness due to a prescribed earthquake motion in the underlying rock. It is evident that certain types of ground favor certain types of structure, but the author is unable in the present state of knowledge to "quantify" this influence. Mr. Eremin's inferences that "rigid frame structures are more resistive to the earthquake shocks" and that "structures based on a rigid slab and soft foundation would be more resistive to the seismic forces than those based on spread footings" can not be proved at the present time in terms of the data of the paper. The paper's section on "Rigid Buildings" referred to brick and concrete buildings with shear walls and of good construction with modern materials.

The author certainly considers the findings of soil mechanics as applicable to the problem, but the paper was deliberately limited in scope to the data obtained in large earthquakes. Such soil data as would be pertinent were largely unavailable, even for the recent United States earthquakes for which U. S. Coast and Geodetic Survey accelerograms have been obtained. A current activity of the Earthquake Engineering Research Institute is directed toward the future procurement of extended instrumental records, along with soil data which would permit the approaches outlined by Mr. Ambraseys.

The soil features which Mr. Ambraseys has discussed would all seem to require and justify research: strength under earthquake loading conditions; positive and negative dynamic pore pressures; post-earthquake soil deformations; and parameters needed for computations of ground response. It is hoped that soil mechanics will embrace these kinds of studies as well as analytical, laboratory, and field studies of the earthquake-ground-structure system. In further qualification of this point, it is important to distinguish between very small soil stresses, which can probably be treated by the usual viscoelastic theories, and stresses in the vicinity of the soil strength. For any practical situation there will be certain geographic areas and certain soils that will be only slightly stressed. In the present state of the art, it is not clear to what extent soils will be stressed into the strength range in earthquakes. In large shocks, such high stressing is probable, in the author's

a. Proc. Paper 1730, August, 1958, by C. Martin Duke.

1. Dept. of Eng., Univ. of California, Los Angeles, Calif.

opinion, particularly in the vicinity of foundations and at soil-rock interfaces. In answer to Mr. Ambrasey's four questions:

I. whether soil strength will be greater under earthquake loading conditions than in the usual static tests cannot be unequivocably answered at present;

II. previously obtained dynamic strength test results do not adequately take account of the stress reversals or the load frequencies associated with destructive earthquakes;

III. studies currently under way at the University of California, Berkeley, by H. B. Seed, will contribute to better understanding relative to (II) above;

IV. the author does not possess data on damping or S-wave velocities at high stresses.

Two corrections should be made to the paper as published in "Proceedings". (1) The bibliographic reference numbers used in the text are correct through number 44; thereafter all numbers should be decreased by one. (2) The provision for "California Schools" on page 11 was effective in 1948, but subsequently the differentiation among ground types has been eliminated.

GLOSSARY OF TERMS AND DEFINITIONS IN SOIL MECHANICS[a]

Discussion by Peter B. Heidema

PETER B. HEIDEMA,[1] F. ASCE.—On page 1826-2 of the Soil Mechanics Journal of October 1958, the Committee on Glossary of Terms and Definitions in Soil Mechanics makes the following two statements:

> "It is therefore recommended that this glossary be published in the Journal of the Soil Mechanics and Foundations Division of the Society, whereby discussion will be invited" (underlining by the writer) and:
>
> "To focus attention on those terms the definitions of which are particularly controversial, it will be noted that some of the terms in the glossary are designated by an asterisk. It is hoped that these terms especially will receive full discussion." (Underlining by the writer).

The writer likes to take advantage of these two "openings" and offers the following, perhaps too lengthy, discussion. Opinions expressed in the discussion are not necessarily those of the employing agency.

With respect to the title it is suggested to insert the word "notations" in the title and to make this: "Glossary of Notations, Terms and Definitions", because the "notations" are the most important item of the Glossary for many people who practice soil mechanics.

Whenever the Glossary is consulted, it will be most often to look up the correct "notation" to be used. For instance, the writer has seldom had occasion to look for a definition but has practically worn out his copy of the 1941 ASCE Soil Mechanics Nomenclature looking for "notations". The writer suggests also to try to split up the glossary into two parts, firstly, a practical part, containing the terms, one term for each subject, and their "notations" and secondly, a more theoretical part in the form of a soil mechanics dictionary containing the terms (all the terms, including those coined and invented to date) and their definitions. This would save considerable time in looking up "notations" by people who do not have the time to leaf through a great number of pages. The writer considers the issuance of the second part optional.

The Glossary appears to be geared too much to people whose job it is to write a good report or a good article for a magazine but it is just too slow for people who work in high pressure organizations as many soil people do. Splitting up the glossary in a list of "notations" and a dictionary would benefit both categories.

It also appears that quite a few terms, which are being used in discussions on the physio-chemical properties of clays, are not in the glossary. A dictionary of just soil mechanics terms would take care of this matter also.

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a. Proc. Paper 1826, October, 1958, by the Committee on Glossary of Terms and Definitions in Soil Mechanics.

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The terms and definitions are, generally speaking, more definitely fixed in a person's memory than the "notations". The main reason for this fact is that they are easier to remember but another reason is that a person dealing in soil mechanics has found in the literature various "notations" for the same term or various terms represented by the same "notations". As this can be quite confusing and somewhat time-consuming, the writer has tried to standardize what he calls the "notations" for lack of a better word or for lack of a word which is or may be in general use but of which he has no knowledge. The writer does not like to use the word "symbol" and the Committee evidently does not like it any more either because it has changed the title from "Soil Mechanics Nomenclature, Tables 1, 2, 3, 4 and 5" to "Glossary of Terms and Definitions in Soil Mechanics". This is a decided improvement over what we had but it is not enough, because it leaves out the most important items as explained above, the "notations".

One of the reasons that the writer has always shied away from the use of the word "symbol" is that every time he tried to use it he had to go through a great deal of explanation to make clear to the newcomer in soil mechanics just what he meant. And if a person looks up in Webster's dictionary the explanation of the word "symbol", it becomes clear why so much explanation is necessary. The essence of what is stated there starts like this: "A symbol is a sign by which one knows or infers a thing". This is not so bad, except for the fact that when one looks up the word, "sign", he finds that it can be among a multitude of other things: "A conventional symbol representing an idea, as a word, letter or mark".

But then, staying with the word "symbol", Webster goes on under 2, as follows: "In writing or printing, a conventional sign, such as a character, a letter, or an abbreviation used instead of a word or words", . . . Here then suddenly are three things that a symbol could be, and neither of the three can really be called a sign. And even if one of the three, say a character, could be called a sign, the other two would still cause, and do cause endless trouble and confusions. So, the writer is very happy to see that the Committee has eliminated the expression "Symbol" from the Glossary and he hopes that it will be for good. In soil mechanics the expression "symbol", if it is used at all, should stand for nothing else but an indicator such as cross-hatchings. Under "notation" one finds in Webster that it is "a marking" and under 1., "Act, process or method of representing by a system or set of marks, signs, figures, or characters; also, any system of symbols or abbreviated expressions used in an art or science (underlining by the writer) to express technical facts, quantities, etc. as, musical notation, mathematical notation". It is for these reasons that the writer likes to introduce the expression "notations" in the title.

The writer is also very happy about the fact that the Committee has adhered to alphabetical order, through thick and thin, as he calls it. This is one more very definite improvement. The writer has gradually come to the conclusion that alphabetical order is all that one needs. Everything additional, including numbers and numbering, are superfluous and oftentimes confusing. Numbering is definitely not a good system because it does not allow for insertions. One can add something at the end only but one cannot add something at the beginning and what is still more embarrassing, one cannot insert anything without upsetting the entire numbering system. For these reasons the writer expresses the hope that the Committee will carry out its intentions and omit the numbering from the final printed work.

Also, the writer would very much like to go one step further and omit all the references starting with "see". These "see" references cause a loss of time and decrease the efficiency of the set-up. It is sort of a nuisance and disappointment when a person, who is trying to look up a "notation", arrives at the spot where it should be according to the alphabetic set-up and is then directed to look somewhere else. For instance when he looks up "C" horizon, the glossary says "See Horizon" instead of telling him what the "C" horizon is right then and there. There are many other instances like this one, which are too numerous to mention. It just means a lot of extra leafing through a lot more pages.

With respect to the explanatory notes, the writer would have preferred to see under "1. Dimensions" an "M" for mass instead of the "F" for force. In his opinion a force is completely defined by the basic dimensions "L", "M", and "T". However, he realizes that "F" for force appears to have gradually crowded out the "M" for mass.

At the end of the "Explanatory Notes" and the beginning of the "Terms and Definitions", the latter should have been announced in some way. As it stands, there does not seem to be any separation between the two subjects.

The writer has deliberated at length just how to handle the discussion of the "Terms and Definitions" and has decided, for better or for worse, to discuss the items in the same order as they stand. He will not mention those items with which he is in complete agreement, regardless of whether they carry an asterisk or not.

With respect to the material under "Terms and Definitions", the writer is of the opinion that too many new terms and "notations" have been coined and invented during the past 35 years with the result that it is becoming increasingly difficult to see the forest through the trees. He suggests to return to the relative simplicity of 35 years ago and to cut out all unnecessary terms and "notations" which duplicated others already in use. These unnecessary terms and "notations" tend to confuse people who are not specialists in soil mechanics and only make the subject look much more complicated than it is. In fact, there are so many terms and "notations" in existence now that even an expert does not know any more which one to use and when. The writer also believes that too many authors have coined and used too many terms and that the time has come for standardization. The writer is too strongly reminded of the days in Europe before normalization came along, when every manufacturer invented his own bolt and thread sizes so a competitor could not use them. It does not make for efficiency and expediency when a reader has to switch from one set of "notations" to another as he goes from one text to another and it becomes downright confusing when one certain "notation" can have two or more different meanings. So, as the discussion progresses, the writer plans to make suggestions towards what he prefers to call standardization.

In addition, the writer likes to make the following comments:

It is suggested to leave a space and to place a capital letter at every change from one letter of the alphabet to the next one.

Of the terms which have been discussed, only those underlined are proposed to be retained.

If the glossary as it stands now is not to be split up in a "Glossary of Terms and 'Notations'" and a "Dictionary of Terms and Definitions", as suggested by the writer, it is proposed that the definition be given in all cases where it cannot be found within one sheet before or after the sheet on which the term appears.

All that has been discussed here has undoubtedly been considered by the Committee, but the writer is of the opinion that it will not do any harm to bring it to the fore. Also, he is aware of the fact that many of his suggestions and proposals do not merit consideration, but he feels that he is entitled to express an opinion anyway.

Whatever the writer has added to the enormous amount of work already done by the Committee, is intended only as suggestions and is so indicated. It is hoped that it will be so considered and not as criticism.

Throughout the Glossary, as presented by the Committee, the writer was impressed by the extreme clarity and remarkably excellent and unmistakable wording of the definitions.

Starting with the letter "A", we then have or could have:

"A"—Horizon

It is suggested to place this term and its definition here.

Allowable Bearing Value (Allowable Soil Pressure)

It is suggested to use the term "Allowable Bearing Value", and the notation "G_a" only, for all future work.

Allowable Pile Bearing Load

It is suggested to reduce the two "notations" to one and preferably to P_a.

Angle of Obliquity

Reduction to one of the four "notations" mentioned is suggested. Also, it is suggested to use "δ" as is already used for the angle of external friction. This should not cause confusion because the angle of external friction is nothing but a special case of "The angle between the direction of the resultant stress or force acting on a given plane and the normal to that plane". The writer cannot see a valid reason to use four additional "notations".

Atterberg Limits, together with a definition of same, should be inserted after "Area Ratio".

"B"—Horizon

Placing this term and its definition here is suggested.

Bar-Linear Shrinkage

BLS

Decrease in one dimension of a bar of soil, expressed as a percentage of the original length, when the moisture content is reduced from the liquid limit to the "shrinkage limit".

Base Exchange

A change of the second word "specie" to "species" is suggested.

Binder (Soil Binder).

It is suggested to delete the term "Binder" and to place the definition under "Soil Binder" under the letter "S".

Bearing Capacity of a Pile

It is suggested to reduce the number of "notations" to one and preferably to P_u, representing the ultimate pile bearing load.

"C"—Horizon

It is suggested to place this term and its definition here.

Centrifuge Moisture Equivalent

CME.

It is suggested to place this term and its definition here and to use the "notation" CME only, for future work.

Clay Size

It is suggested to change this to Clay Sizes.

Cobble (Cobblestone)

It is suggested that the definitions for "rounded" and "semi-rounded" be included in the glossary, if such definitions do exist.

Coefficient of Absolute Viscosity

It is suggested to explain right here.

Coefficient of Active Earth Pressure

It is suggested to omit (see below).

Coefficient of Compressibility (Coefficient of Compression)

It is suggested to use the term "Coefficient of Compressibility" only, for all future work. Also, the writer would prefer to insert the expression $\frac{e_1 - e_2}{p_2 - p_1} = \frac{de}{dp}$ and to call it the tangent of the angle, which the pressure-void ratio curve, drawn to the natural scale, makes with the positive x-axis, this in preference to the secant slope. The writer prefers Prof. Taylor's simple expressions on page 225 of his book.

Coefficient of Consolidation

It is suggested to change "3" in the formula to "e".

Coefficient of Earth Pressure, Active, At Rest, Passive

It is suggested to distinguish only three items: The Coefficient of Active Earth Pressure K_A, the Coefficient of Earth Pressure at Rest to K_O, and the Coefficient of Passive Earth Pressure Kp. Also, to change the letter p in K_p to a capital "P".

Coefficient of Permeability (Permeability)

It is suggested to omit "(Permeability)" and to change "(usually 20° C)" back to "(usually 10° C)".

Coefficient of Subgrade Reaction (Modulus of Subgrade Reaction)

It is suggested to use the term "Modulus of Subgrade Reaction" only, for all future work, with the "notation" "k" and to move it to "Modulus of Subgrade Reaction" under the letter "M". The modulus could be defined as "the tangent of the angle which a line, passing through the origin of a load versus settlement plot and the point representing the load for 0.05" settlement, makes with the positive x-axis. The load-settlement plot represents the results of a plate load test, using a 30-inch or greater diameter loading plate".

Coefficient of Viscosity

If the coefficient of absolute viscosity is intended to be the very same thing as the coefficient of viscosity, it is suggested to replace the parentheses by an equal mark. If the coefficient of absolute viscosity is not intended to be exactly the same thing as the coefficient of viscosity, it should be listed separately. If it is, it seems that it could be omitted. It is further suggested to insert here the coefficient of kinematic viscosity. See definition of the term "Soil Mechanics", and page 7 of the 1941 Soil Mechanics Nomenclature.

Coefficient of Volume Compressibility (Modulus of Volume Change)

It is evident that what is meant here is what Professor Terzaghi called "the coefficient of volume change", either decrease or increase. The writer does not understand the necessity for coining two additional expressions and suggests to drop both and to continue to call this the coefficient of volume change. It is further suggested to continue to call the original void ratio "e_0" and not just "e".

Compaction Curve (Proctor Curve) (Moisture-Density Curve)

It is evident that these expressions stand for exactly the same thing. It is therefore suggested to replace the parentheses by equal marks or, better yet, to simplify things by dropping two of the expressions and to settle for one, preferably the first one, because it has two syllables and because it is about as expressive as it could be. It is further suggested to delete the word "density" from the definition and to replace "water" in water content by "moisture". The expression water content reminds the writer too much of water contained in a bucket and certainly water does not occur in such concentrated form in a normal soil.

Compaction Test (Moisture-Density Test)

It is suggested to reduce the number of terms from two to one and preferably to "Compaction Test", for all future work.

Compression Index

It is suggested to change the definition as follows: "The tangent of the angle between the linear portion of the pressure-void ratio curve drawn on a semi-log plot with the positive direction of the x-axis". This expression is believed to be more unmistakable and automatically takes care of the minus sign.

Compressive Strength (Unconfined Compressive Strength)

It is believed that it is generally understood that compressive strength and unconfined compressive strength are not identical. If they are to be considered identical, the parentheses should be replaced by an equal mark. If they are not to be considered identical they should be listed individually and explained individually. The writer feels that they are not identical and that compressive strength has a more general meaning such as the compressive strength of steel and concrete. He, therefore, suggests to eliminate the expression "compressive strength" and also the "notation" "p_c" for this term, because it could be mistaken for preconsolidation pressure.

Consistency Index

It is suggested to define it here and not under "Relative Consistency". It is proposed to use the "notation" "CI", and the "notation" "I" for all indexes.

Consolidated-Drained Test (Slow Test)

It is suggested to eliminate "(Slow Test)" in all future work and accordingly in this glossary and to introduce the "notation" "CD" preceding test.

Consolidated-Drained Direct Shear Test

CDD.

This "notation" is suggested as an insert.

Consolidated-Drained Triaxial Compression Test

CDT.

It is suggested to insert this "notation".

Consolidated-Undrained Test (Consolidated Quick Test)

It is suggested to eliminate "(Consolidated Quick Test)" in all future work and to introduce the notation "CU" preceding test. It is further suggested to change "triaxial test" to "triaxial compression test" and "water content" to "moisture content".

Consolidated-Undrained Direct Shear Test

CUD.

This "notation" is suggested as an insert.

Consolidated-Undrained Triaxial Compression Test

CUT.

This "notation" is suggested as an insert.

Initial Consolidation (Initial Compression)

It is suggested to omit "(Initial Compression)", for all future work.

Primary Consolidation (Primary Compression) (Primary Time Effect)

It is suggested to retain only the expression "Primary Consolidation", for all future work.

Secondary Consolidation (Secondary Compression) (Secondary Time Effect)

It is suggested to retain only the expression "Secondary Consolidation", for all future work.

Consolidation-Time Curve (Time Curve) (Consolidation Curve) (Theoretical Time Curve)

It is suggested to retain only the expression "Consolidation-Time Curve" and to change this expression to "Consolidation Versus Time Curve". To place the words "Consolidation" and "Time" in the order shown is just another novelty because these curves always were called time-consolidation curves. The writer can see no pertinent reason to reverse the order of the words "Time" and "Consolidation" unless the word "Versus" is inserted. It is further suggested to change the expression "degree of consolidation" to "amount of consolidation" as the degree of or the percentage of consolidation is seldom, if ever, plotted any more.

Critical Circle (Critical Surface)

It is suggested to split this item in two and to give a definition for each because they are two different things. For "Critical Circle" the writer suggests the following definition: "The sliding circle assumed in a two-dimensional slope stability analysis for which the factor of safety is a minimum".

Critical Hydraulic Gradient

It is suggested to define right here just what it is and to give the "notation".

Critical Surface

The reader is referred to the remarks made under "Critical Circle".

Critical Void Ratio

It is suggested to state right here what it is and to give the "notation".

Deflocculating Agent (Deflocculant) (Dispersing Agent)

It is suggested to continue the use of the term "Deflocculating Agent" only, for future work.

Degree of Consolidation (Percent Consolidation)

It is suggested to use, for future work, the term "Percent Consolidation" only, because it is more expressive, and to list it, together with its "notation", under the letter "P". The definition is to remain the same.

Degree of Saturation

It is suggested to delete this expression and to use only "Percent Saturation" with the one word-spelling of Percent.

Degree of Sensitivity (Sensitivity Ratio)

It is suggested to use, for future work, the term "Sensitivity Ratio" only, because it is more expressive, and to move it, together with its "notation", to the letter "S". The definition of it should perhaps be stated right here. Also, the reference to "Remolding Index" appears to be a misnomer and should probably have been "Remolding Sensitivity".

Density

The expression "Density" by itself should never be used in soil mechanics, according to the writer, because its use has led to endless confusion in soils laboratories, on construction jobs and in all sorts of literature, technical magazines in particular. The writer suggests to delete this term, standing by itself, from the glossary.

Deviator Stress

It is suggested to use the "notation" "6" only and to eliminate all other possible "notations" for this term in the future.

Dispersing Agent

It is suggested to delete this term, as mentioned before.

Dry Unit Weight

It is suggested to move this term and its "notation" to "Unit Weight" under the letter, "U". The writer suggests to drop the term dry density, even though it is shorter, and prefers to use the term "Unit Dry Weight" over "Dry Unit Weight". He considers the term "Dry Unit Weight" another one of these novelties that only tend to confuse the issue and that have little if any betterment value attached to them.

Earth

The writer suggests to place the explanation right here, or, better yet, to omit this term altogether.

Earth Pressure

Active Earth Pressure

It is suggested to insert "Total Pressure" after "P_A" and "Unit Pressure" after "p_A".

At Rest Earth Pressure

It is suggested to insert "Total Pressure" after "P_O" and "Unit Pressure" after "p_O".

Passive Earth Pressure

It is suggested to insert "Total Pressure" after "Pp" and "Unit Pressure" after "p_p". Also, it is suggested to change "p_p" to "p_P".

It is suggested to insert the term "Electro-Osmosis" and its definition after "Elastic State of Equilibrium."

Effective Diameter (Effective Size), D_{10}, D_e.
It is suggested to discontinue the use of "Effective Diameter" and of "D_e".

Effective Drainage Porosity
It is suggested to define it right here, if it is necessary to retain this term, and to place "n_e" here.

Effective Porosity (Effective Drainage Porosity)
It is suggested to delete the term "Effective Porosity" for the future.

Effective Size
It is suggested to place the definition and the "notation" here. (See above).

Equivalent Diameter (Equivalent Size)
It is suggested to use the first expression for the future, and to delete "Equivalent Size".

Equivalent Fluid
There appears to be a typographical error in the last line of the definition.

Excess Hydrostatic Pressure
It is suggested to place the definition and the "notation" here.

Failure by Rupture
It is suggested to place the definition here, at least if it is considered necessary to retain this term. The writer would not miss it if it were deleted.

Field Moisture Equivalent
It is suggested to define this term and to give the "notation" "FME" here.

Filter (Protective Filter)
It is suggested to use only the second term, for future work, because it fits the definition and to place it under the letter "P". This would result in the elimination of one more term.

Flocculent Structure
It is suggested to explain the term here.

Flow Index
It is proposed to use the "notation" "FI" only, for future work, thus eliminating all other "notations". Also, it is proposed again to use the letter "I" for all indexes and to qualify it by a prefix.

Free Water Elevation (Water Table) (Ground Water Surface) (Free Water Surface) (Ground Water Elevation)
It is suggested to retain the term "Water Table" only, for future work, and to place it and its definition under the letter "W".

Gravel
The writer still favors retaining the #10 sieve or the 2 mm. size as the lower limit for gravel sizes and further suggests to name this term "Gravel Sizes", as was done with the "Clay Sizes".

Height of Capillary Rise
It is suggested to eliminate this term and to use the term "Capillary Rise" only, for future work.

Honeycomb Structure
It is suggested to define the term here.

Horizon (Soil Horizon)

It is suggested to delete the term "Horizon" and to place "Soil Horizon" and its definition under the letter "S".

"A" Horizon

Placing this term and its definition under the letter "A", as stated above, is suggested.

"B" Horizon

Placing this term and its definition under the letter "B", as stated above, is suggested.

"C" Horizon

Placing this term and its definition under the letter "C", as stated above, is suggested.

Hydraulic Gradient

It is suggested to use the "notation" "i" only, for future work.

Excess Hydrostatic Pressure (Hydrostatic Excess Pressure)

It is suggested to use the term "Excess Hydrostatic Pressure" and the "notation" "u" only, for all future work. It is further suggested to place this term under the letter "E". The reader is referred to the discussion under "Excess Hydrostatic Pressure".

Hygroscopic Capacity (Hygroscopic Coefficient)

Use of the term "Hygroscopic Coefficient" and the "notation" "w_h" only is suggested for future work.

Influence Value

It is proposed to use the "notation" "V_i", for future work.

Initial Consolidation (Initial Compression)

It is suggested to omit the second term for future work, as stated before.

Laminar Flow (Stream Line Flow) (Viscous Flow)

It is suggested to retain the term "Laminar Flow" only, for future work.

Ledge

It is suggested to place the definition here.

Line of Seepage (Seepage Line) (Phreatic Line)

It is suggested to use the third term only, for all future work, because it is the only term that fits the definition, and to list it, together with its definition, under the letter "P".

Linear Expansion

It is proposed to use the "notation" "LE" instead of "L_E".

Liquid Limit

It is suggested to use the "notation" "LL" only, for all future work.

Liquidity Index (Water-Plasticity Ratio) (Relative Water Content)

It is suggested to use the first term and the "notation" "LI" only, is proposed for all future work.

Loam

It is suggested to insert in the definition that it is a portion of the "A" horizon.

Modulus of Elasticity (Modulus of Deformation)

It is suggested to use the term "Modulus of Deformation" and the "notation" "M" only, for all future work, because few soils perform elastically and then only under certain conditions. This idea would eliminate one more term and one more "notation". It is further suggested to omit the idea of a secant because a certain modulus of deformation is generally good for only one point on the stress-strain curve.

Modulus of Subgrade Reaction

This term has been discussed under "Coefficient of Subgrade Reaction".

Modulus of Volume Change

This term has been discussed under "Coefficient of Volume Compressibility".

Mohr Envelope

It is suggested to use this term only, for all future work.

Moisture Content (Water Content)

As discussed and explained before, the writer much prefers the term "Moisture Content" over "Water Content" for soils. It is suggested to change the term "water content", wherever it appears, to the term "moisture content".

Moisture Density Curve

It is suggested to delete this term, for future work.

Moisture Density Test

It is suggested to delete this term, for future work.

Moisture Equivalent

It is suggested to delete this term, for future work.

Centrifuge Moisture Equivalent

This term has been discussed under the letter "C".

Field Moisture Equivalent

This term has been discussed under the letter "F".

Neutral Stress and Normal Stress

If these terms are to be placed in the glossary here, it is suggested that also the terms neutral pressure and normal pressure be inserted here and that they all be defined here and their "notations" given.

Optimum Moisture Content (Optimum Water Content)

The writer much prefers the term "Optimum Moisture Content" for reasons stated before and suggests to delete the second term, for all future work. It is further suggested to use the "notation" "OMC" only, for future work, because W_o or w_o could be mistaken for original weight or original moisture content. Also, the letter "o" is too often used to indicate something that is zero, like zero load.

Organic Clay

If the term "high" organic content is to be used, it would seem advisable to set a lower limit on it, say a certain percentage of the dry weight.

Organic Silt

What has been stated under "Organic Clay" pertains to this term also.

Organic Soil

What has been stated under "Organic Clay" goes for this term also.

Overburden Pressure

It is suggested to insert this term here with its definition and "notation".

Passive Earth Pressure

It is suggested to give the definition and the "notation" here, or to repeat them here, so that a person does not have to go all the way back to "Earth Pressure", as it stands now, to find what he is looking for. This has been stated before under "Earth Pressure" c.a.

Penetration Resistance (Standard Penetration Resistance) (Proctor Penetration Resistance)

It is suggested to use the first term and the "notation" P_R only, for future work, and to re-arrange "a", "b", and "c" to the effect that "c" comes first and "a" last, because the term was probably originated by Proctor.

Penetration Resistance Curve (Proctor Penetration Curve)

It is suggested to delete the second term, for future work.

Percent Consolidation

This term was discussed under "Degree of Consolidation".

Percent Saturation (Degree of Saturation)

It is suggested to eliminate the second term, for future work.

It is suggested to insert the expression "Percent Voids" here with its "notation" and its definition.

Permeability

It is suggested to omit this term because sufficient information is already given under "Coefficient of Permeability".

Phreatic Line

It is suggested to place the definition here, as discussed before under "Line of Seepage".

Phreatic Surface

It is suggested to place the definition here.

Phreatic Water

It is suggested to place the definition here.

Piezometric Surface

It is suggested to add "Phreatic Surface".

Plastic Limit and Plasticity Index

It is suggested to use the "notations" "PL" and "PI" only, for future work.

Pore Pressure (Pore Water Pressure)

It is suggested to eliminate these terms and to use only the term "Neutral Pressure", and to place this term under "Stress" and the letter "S".

Preconsolidation Pressure (Prestress)

It is suggested to use the term "Preconsolidation Pressure" only, for future work.

Pressure-Void Ratio Curve (Compression Curve)

It is suggested to use the term "Pressure-Void Ratio Curve" only, for future work, because the term "Compression Curve" could mean any kind of a compression curve.

Primary Consolidation (Primary Compression) (Primary Time Effect)
It is suggested to use the term "Primary Consolidation" only, for all future work, and to place the definition here.

Principal Stress
It is suggested to place the definition and the "notations" here, since the definitions of "Principal Planes" are already here.

Proctor Compaction Curve
It is suggested to omit this term. The reader is referred to the discussion under "Compaction Curve". It is noted that at present the term "Proctor Compaction Curve" is not mentioned under "Compaction Curve".

Proctor Penetration Curve
It is suggested to delete this term. The reader is referred to the discussion under "Penetration Resistance Curve."

Proctor Penetration Resistance
It is suggested to delete this term. The reader is referred to the discussion under "Penetration Resistance".

Profile
It is suggested to delete this term. The reader is referred to the discussion under "Soil Profile".

Protective Filter
It is suggested to place the definition here. The reader is referred to the discussion under "Filter".

Pumping of Pavement (Pumping)
It is suggested that this term be deleted. It is sufficiently defined under Pavement Pumping.

Quick Test
It is suggested to delete this term, for all future work. The reader is referred to the discussion under "Unconsolidated Undrained Test".

Radius of Influence of a Well
It is suggested to place a "notation" here and to make it "r_i."

Relative Consistency
It is suggested to delete this term, for future work. The reader is referred to the discussion under "Consistency Index".

Relative Water Content
It is suggested to delete this term, for future work. The reader is referred to the discussion under "Liquidity Index".

Remolding Index
It is proposed to use the "notation" "RI".

Remolding Sensitivity (Sensitivity Ratio)
It is suggested to retain the term "Sensitivity Ratio" only, for future work, and to list it, together with its "notation", under the letter "S". The "notation" "SeR" is proposed.

Rupture Envelope (Rupture Line)
It is suggested to delete both these terms. The reader is referred to the discussion under "Mohr Envelope".

Rock Flour
If this term is supposed to be the same as "Silt", as is indicated under "Silt", the definition should be placed here also. The reader is referred to the discussion under "Silt".

Safety Factor
It is proposed to insert this term here with its definition and to use the "notation" "SF".

Sand
The writer is still in favor of using the 10-mesh sieve and the 2 mm. size as the upper limit for sand. Also, he suggests to change the term to "Sand Sizes" and "particles of rock" to "particles", because, generally, it will never be known just what the particles consisted of.

Saturated Unit Weight
The writer prefers the term "Unit Saturated Weight", which has been in use for at least 30 years and can see no valid reason for the change.

Saturation Curve
It is suggested to place the definition here as follows: "A plot of the unit dry weight of a soil against its moisture content at 100 percent saturation". The reader is referred to the discussion under "Percent Saturation".

Secondary Consolidation (Secondary Compression) (Secondary Time Effect)
It is suggested to retain only the first term, for future work, and to list the definition here, as discussed under "Consolidation".

Seepage Line
It is suggested to delete this term, for future work, as was discussed under "Line of Seepage".

Seepage Velocity
It is suggested to retain only the "notation" "v_s", for future work.

Shear Failure (Failure by Rupture)
It is suggested to retain the first term only, for future work.

Shear Stress (Shearing Stress) (Tangential Stress)
It is suggested to retain the term "Shear Stress" and the "notation" "γ" only, for future work.

Shrinkage Ratio
The definition of this term has caused writer a good deal of trouble when explaining it, defending it and using it, though not often. If this term and this concept is to be retained in soil mechanics, it is suggested to express the volume change as a percentage of the original volume, for future work. This would bring it in line with the definition of the linear shrinkage. Also, if this concept is to be retained, the "notation" "SR" is proposed for future work. Rarely has the writer had occasion to use this concept and he is not aware of the value of it.

Silt (Inorganic Silt) (Rock Flour)
It is suggested to delete these three terms. The reader is referred to the discussion under "Rock Flour".

Silt Size
It is suggested to change to "Silt Sizes".

Single-grained Structure

It is suggested to dispense with this term here.

Slope Angle

β.

It is suggested to insert this term here and to define it as follows: "Angle between the slope of the soil or of a dam and the positive direction of the x-axis".

Slow Test

It is suggested to delete this term, for all future work. The reader is referred to the discussion under "Consolidated-Drained Test."

Soil (Earth)

It is suggested to delete the term "Earth". The reader is referred to the discussion under "Earth".

Soil Binder

It is suggested to place the definition here. The reader is referred to the discussion under "Binder".

Soil-forming Factors

It is suggested to include chemical and physical activity.

Soil Horizon

It is suggested to place the definition here. The reader is referred to the discussion under "Horizon".

Soil Profile (Profile)

It is suggested to delete the term "Profile", for future work. The reader is referred to the discussion under "Profile".

Soil Texture

It is suggested to repeat the definition given under "Gradation" here. It is noted that the term "Soil" Texture has been used under "Gradation". If quotes are to be applied to the word "Soil" in some places should not they be applied in all other places?

Specific Gravity

It is suggested to omit this term, standing all by itself.

Specific Gravity of Solids

It is suggested to use the "notation" "G" as in the past, and no other one. The writer fails to see why a capital letter without a subscript could not be used.

Apparent Specific Gravity

It is suggested to state: Specific Gravity, Apparent, and to use only the "notation" "G_a", for future work.

Bulk Specific Gravity

It is suggested to state: Specific Gravity, Bulk, and it is proposed to use only the "notation" "G_b" and to delete "G_m", "S_m" and the term "Specific Mass Gravity." The writer considers this to be another case of coined terms which are seldom used in practical soil mechanics.

Specific Surface

It is suggested to give this term a "notation". The "notation" "S_s" is proposed.

Stability Factor (Stability Number)

Since two eminent men in soil mechanics have established two different stability criteria, both of which have been used extensively, it is wellnigh impossible to delete one of these at this stage. It is therefore suggested to list both and to give both equal weight, and to mention the name of the originator with each.

Stabilization

It is suggested to delete this term.

Standard Compaction

It is suggested to delete this term, or, if it is to be retained to define it. This has not been done under "Compaction Test" where the reader is referred to.

Standard Penetration Resistance

It is suggested to delete this term. It only adds to the confusion. The reader is referred to the discussion under "Penetration Resistance".

Sticky Limit

The "notation" "w_{st}" instead of "T_w" is suggested.

Stream Line Flow

It is suggested to delete this term, for future work. The reader is referred to the discussion under "Laminar Flow".

Stress

It is suggested to use the "notation" "σ" only, for future work. Similarly, it is suggested to use the "notation" "σ" only, for the terms "Effective Stress", "Effective Pressure" and "Intergranular Pressure" and the "notation" "u" only, for "Neutral Stress", "Pore Pressure" and "Pore Water Pressure". It is proposed to add the term "Neutral Pressure" after "Neutral Stress".

Normal Stress

It is proposed to add the term "Normal Pressure" and to use the "notation" "σ_n" only, for future work.

Shear Stress (Shearing Stress) (Tangential Stress)

It is suggested to delete this term here and to list it under "Shear Stress". The reader is referred to the discussion under "Shear Stress".

Structure

It is suggested to delete this term, because it has already been taken care of under "Soil Structure".

Subgrade Surface

It is suggested to delete this term because it has been sufficiently taken care of under "Subgrade".

Submerged Unit Weight

Deletion of this term here is suggested.

Tangential Stress

It is suggested to delete this term, for future work. The reader is referred to the discussion under "Shear Stress".

Theoretical Time Curve

It is suggested to delete this term, for future work. The reader is referred to the discussion under "Consolidation-Time Curve".

Till

It is suggested to delete this term, for future work. The reader is referred to the discussion under "Glacial Till".

Time

It is suggested to insert this term and its notation "t".

Time Curve

It is suggested to delete this term, for future work. The reader is referred to the discussion under "Consolidation-Time Curve".

Time Factor

It is suggested to retain the "notation" "T_v" only, for future work. Also, it is suggested to multiply the value of "T" by 4 so that the formula will always apply to the total thickness of a double-drained layer. This would seem more practical.

Total Stress

It is suggested to delete this term here, as it is already well taken care of under "Stress".

Toughness Index

It is suggested to use only one "notation", for future work and the "notation" "TI", like "PI" is used for plasticity index, is proposed.

Triaxial Shear Test (Triaxial Compression Test)

It is suggested to use the term "Triaxial Compression Test" only, for future work.

Ultimate Bearing Capacity

It is suggested to use only one "notation" for future work and it is proposed to use q_{ult}. The reader is referred to the discussion under "Bearing Capacity of a Pile".

Unconfined Compression

It is suggested to insert this term here and it is proposed to use the "notation" "UC".

Unconfined Compressive Strength

It is suggested to place the definition and the "notation" here and to delete the term "Compressive Strength" in soil mechanics. The reader is referred to the discussion under "Compressive Strength".

Unconsolidated-Undrained Test (Quick Test)

It is suggested to delete the term "Quick Test", for future work, and to use the "notations" "UUD" for the unconsolidated-undrained direct shear test and "UUT" for the unconsolidated-undrained triaxial compression test. The reader is referred to the discussions under "Consolidated-Drained Test" and "Consolidated-Undrained Test".

Dry Unit Weight (Unit Dry Weight)

It is suggested to continue to use the term "Unit Dry Weight" and the "notation" "γ_d" only, for future work.

Saturated Unit Weight

It is suggested to continue to use the term "Unit Saturated Weight". The "notation" "γ_s" is proposed. The writer fails to see the necessity for using more than one letter where just one will do, when no confusion is possible.

Submerged Unit Weight (Buoyant Unit Weight)

It is suggested to continue the use of the term "Unit Buoyant Weight" and the "notation" "γ_b" is proposed.

Wet Unit Weight (Mass Unit Weight)

It is suggested to continue the use of the term "Unit Wet Weight" and the "notation" "γ" is proposed. The writer has difficulty to agree to waste the "notation" "γ" on the term "Unit Weight" because the concept "unit weight" by itself is seldom or never used. Furthermore, it is suggested to delete the term "Mass Unit Weight", for future work.

Zero Air Voids Unit Weight

It is suggested to delete this term, its "notation" and its definition, for future work, as it is synonymous with "unit saturated weight" for all practical purposes.

Virgin Compression Curve

It is suggested to place a definition here. As it stands, the reader is referred to "Compression Curve" near the beginning of the glossary. From there he is referred to "Pressure-Void Ratio Curve" in the last part of the glossary. Once there, the reader does not find a definition.

Viscous Flow

It is suggested to dispense with this term, for future work, because all flow is viscous to some extent. If it is to be used, it would seem that a lower limit would have to be attached to the viscosity of the flow.

Critical Void Ratio

It is suggested to move this term in with all the other "Critical" terms under the letter "C". The reader is referred to the discussion under "Critical Void Ratio".

Volumetric Shrinkage (Volumetric Change)

It is suggested to retain the term "Volumetric Shrinkage" only, for future work and to change "expressed as a percentage of the 'soil' mass when dried" to "expressed as a percentage of the original volume". Also, the "notation" "VS" is proposed.

Water Content

It is suggested to change "water" to "moisture" for future work. The reader is referred to the discussion on page 11 under "Compaction Curve".

Water-Plasticity Ratio (Relative Water Content) (Liquidity Index)

It is suggested to retain the term "Liquidity Index" only, for future work, and to place it under the letter "L". The reader is referred to discussion under "Liquidity Index".

Water Table

It is suggested to place the definition here as follows: "Depths or elevations at which the pressure in the water is zero with respect to the atmospheric pressure". Reference is made to the discussion under "Free Water Elevation".

Wet Unit Weight

It is suggested to delete this term as it is amply taken care of under "Unit Weight" and because a person is not apt to look under "Wet" when he is looking for a certain kind of weight or unit weight.

Zero Air Voids Curve (Saturation Curve)

The term has been used for so many years that it would be difficult to dispense with it now. The writer would prefer to use the term "Saturation Curve" only, for future work.

Zero Air Voids Density (Zero Air Voids Unit Weight)

It is suggested to delete these two terms, for all future work, as they will be seldom or never needed. Reference is made to the discussion under "Zero Air Voids Unit Weight".

The discussion of soil mechanics paper 1826 ends herewith.

As an example of what might be accomplished by simplification, the writer proposes the following "List of Terms and Notations", which is based on the foregoing discussion.

SUGGESTED LIST OF TERMS AND "NOTATIONS" FOR USE IN SOIL MECHANICS

Term	Notation	Term	Notation
A		Coefficient of Earth Pressure, at Rest	K_0*
Adhesion, Unit	c_a	Coefficient of Earth Pressure, Passive	K_P*
Adhesion, Total	C_a		
Air-Space Ratio	G_a	Coefficient of Internal Friction	$\tan\phi$
Air-Void Ratio	G_v		
Allowable Bearing Value	q_a	Coefficient of Permeability	k
Allowable Pile Bearing Load	P_a	Coefficient of Uniformity	C_u
Angle of External Friction	δ	Coefficient of Viscosity	μ
Angle of Internal Friction	ϕ	Coefficient of Viscosity, Kinematic	ν
Angle of Obliquity	δ*	Coefficient of Volume Change	m_v
Angle of Repose	α	Cohesion	c
Area of Influence of a Well	a	Compression Index	C_c
Area Ratio of a Sampler	A_r	Concentration Factor	n
B		Consistency Index	CI*
Bar-Linear Shrinkage*	BLS*	Consolidated-Drained	CD*
Bearing Capacity of a Pile	P_u*	Consolidated-Drained Direct Shear	CDD*
C			
California Bearing Ratio	CBR	Consolidated-Drained Triaxial Compression	CDT*
Capillary Head	h	Consolidated-Undrained	CU*
Capillary Rise	h_c	Consolidated-Undrained Direct Shear	CUD*
Centrifuge Moisture Equivalent	CME	Consolidated-Undrained Triaxial Compression	CUT*
Coefficient of Compressibility	a_v	Consolidation Ratio	U_z
Coefficient of Consolidation	c_v	Contact Pressure	p
Coefficient of Earth Pressure, Active	K_A	Critical Height	H_c

I

Term	Notation	Term	Notation
C Continued		Hygroscopic Moisture Content	w_H
Critical Hydraulic Gradient	i_c	I	
Critical Void Ratio	e_c	Influence Value	V_i*
D		J	
Deviator Stress	σ		
Discharge Velocity	v	K	
E			
Earth Pressure, Active, Unit	p_A	L	
Earth Pressure, Active, Total	P_A	Linear Expansion	LE*
Earth Pressure, At Rest, Unit	p_O	Linear Shrinkage	LS*
Earth Pressure, At Rest, Total	P_O	Liquid Limit	LL
Earth Pressure, Passive, Unit	p_P*	Liquidity Index	LI*
Earth Pressure, Passive, Total	P_P	M	
Effective Drainage Porosity	n_e	Modulus of Deformation	M
Effective Size	D_{10}	Modulus of Subgrade Reaction	k
Equivalent Diameter	D	Moisture Content	w
Excess Hydrostatic Pressure	u	N	
F			
Field Moisture Equivalent	FME	O	
Flow Index	FI*	Optimum Moisture Content	OMC
Flow Value	N_ϕ	P	
Freezing Index	F	Penetration Resistance, Unit	p_R
H			
Hydraulic Gradient	i		
Hydrostatic Pressure	u_o		
Hygroscopic Coefficient	w_h*		

II

Term	Notation	Term	Notation
P Continued		Sensitivity Ratio	S_eR
Penetration Resistance, Total*	P_R^*	Shear Strength	s
Percent Consolidation	U	Shear Stress	τ
Percent Saturation	S	Shrinkage Index	SI
p_H	p_H	Shrinkage Ratio	SR*
Plastic Limit	PL	Skin Friction	f
Plasticity Index	PI	Slope Angle*	β^*
Porosity	n	Specific Gravity, Solids	G^*
Preconsolidation Pressure, Unit	p_c	Specific Gravity, Apparent	G_a
Preconsolidation Load*	P_c^*	Specific Gravity, Bulk	G_b
Pressure, Unit	p	Specific Surface	S_s^*
Pressure, Total*	P^*	Stability Factor	N_s
Principal Stress, Major	σ_1	Stability Number	N^*
Principal Stress, Intermediate	σ_2	Sticky Limit	w_{st}^*
		Strain	ε
Principal Stress, Minor	σ_3	Stress, effective	$\bar{\sigma}$
Q		Stress, neutral	u
		Stress, normal	σ_n
R		Stress, total	σ^*
Radius of Influence of a Well	r_i^*	T	
Relative Density	D_d	Time*	t^*
Remolding Index	RI*	Time Factor	T_v^*
S		Toughness Index	TI*
Safety Factor*	SF*	U	
Seepage Force	J	Ultimate Bearing Capacity, Unit	q_{ult}
Seepage Velocity	v_s		

III

Term	Notation	Term	Notation
U Continued		V	
Ultimate Bearing Capacity, Total	Q_{ult}	Void Ratio	e
Unconfined Compression*	UC*	Volumetric Shrinkage	VS*
		W	
Unconfined Compressive Strength, Unit	q_u	Wall Friction	f'
Unconsolidated-Undrained	UU*		
Unconsolidated-Undrained Direct Shear	UUD*		
Unconsolidated-Undrained Triaxial Compression	UUT*		
Unit Weight, Buoyant	γ_b*		
Unit Weight, Dry	γ_d		
Unit Weight, Effective	γ_e		
Unit Weight, Saturated	γ_s*		
Unit Weight, of Water	γ_w		
Unit Weight, Wet	γ *		
Uplift, Unit	u		
Uplift, Total	U		

NOTES:

1. * indicates a change or addition, as suggested by the writer.

2. It is suggested that the list be checked and brought up-to-date after each International Soil Mechanics Conference.

IV

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DESIGN AND CONSTRUCTION OF THE AMBUKLAO ROCK FILL DAM[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] F. ASCE.—The authors have made a notable contribution to the design and construction of rockfill dams with their treatment of the 430 foot high central core Ambuklao Dam.

Having visited the site during the period of initial investigations, the significance of the topographic conditions and, to a somewhat lesser degree, the geologic features was apparent. Thus the authors' observations on the site geology are welcomed as a penetrating statement of the distinctive differences between the geologic conditions quite commonly found at damsites in the tropics and in temperate regions. Justification for the paragraph "The geologic conditions at damsites in the tropics - - are more familiar" is abundantly afforded by Ambuklao geologic features as well as those at many other tropical sites.

The authors have taken full advantage of the topographic situation in utilizing the additional head of about 60 feet in connection with diversion through tunnel "C", and perhaps more significantly in utilizing 180 feet additional gross head (with respect to the normal 505 feet) made possible by the 7200 feet tail tunnel. Could this head have been developed any other way at anything like the cost achieved?

It is interesting to note that "Based on the evidence from these borings and surface observations, it was concluded that the rock in the abutments and in the river bed was not sufficiently strong to withstand the pressure which would be developed by any economically feasible type of concrete dam". Foundation conditions and availability of economically-utilizable core and shell materials are the two principal reasons rock and earthfill dams are likely to be more and more widely used in relatively undeveloped tropical countries as well as elsewhere.

The authors observe that "Except for the foundation of the core, the existing river gravel was left in place". Could they state the approximate maximum thickness?

The authors' crushing tests on the samples of broken rock from the quarry areas are particularly interesting, having demonstrated the diorite was subject to greater breakdown than the metamorphics during blasting, handling and placing in position, contrary to expectation from visual inspection. This demonstrates the value of adequate tests on all components of a rock fill dam.

Although no reference thereto is apparent from the text, could the authors advise if any attempt was made to determine the pore pressures in the three field test fills?

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a. Proc. Paper 1864, December, 1958, by E. Montford Fucik and Robert F. Edbrooke.

1. Chf. Engr., Power Dept., Aluminium Laboratories, Ltd., Montreal, Canada.

The design studies which led to the use of a rockfill with vertical core rather than a sloping one demonstrate clearly conditions which may sometimes be overlooked; i.e. length of seepage path due to increased contact area between the core and the foundation, and increased loading on the contact area with probable minimization of leakage at the contact. Could the authors state the relative weight given these two factors as compared with the saving in fill materials?

The vertical settlement and downstream deflection have been most moderate, at less than 6 inches and about 3 inches respectively. How does the vertical settlement compare with that predicted from tests such as shown by Fig. 7?

It is to be hoped National Power Corporation engineers will, in due course, make available to the profession continuing observations on performance of the outstanding Ambuklao rockfill dam and particularly on vertical settlement and downstream deflection.

A REVIEW OF THE ENGINEERING CHARACTERISTICS OF PEAT[a]

Discussion by E. T. Hanrahan

E. T. HANRAHAN.[1]—Mr. MacFarlane's paper provides a timely review of the present position of work done on the important subject of the engineering characteristics of peat. His paper demonstrates how much work remains to be done, and the many serious gaps in, and the sometimes contradictory nature of the little information which is available.

He has taken a valuable first step towards a standardized nomenclature by including a glossary of terms. It is to be hoped that this paper will lead to a recommended code of practice for laboratory and site determinations of the properties of peat.

Mr. MacFarlane might perhaps with profit have emphasized a little more the fact that peat is, in so many respects, quite different from other soil mechanics materials. Even the simplest routine laboratory tests require modification when testing peat. For example, an analysis of the water contents of samples of peat from a deposit will show widely varying results unless a correction is made for the inorganic content of the specimens, a minute quantity of which can exercise a profound effect on the water content of the whole.

The negligible value of the submerged density of peat is unique and is of very considerable practical importance in any study involving fluctuation of the water-table level. Peat is influenced to a marked degree by its stratification in the field. For instance, by comparing such properties as density, voids ratio, and water content of two specimens of similar peat subject to slight variations of thickness of inorganic overburden, say within a range of 1' to 2', the effect of the variation in stratification may be clearly observed. Thus the not uncommon practice of relating such properties of peat as water content or voids ratio to a peat type without reference to the effective stress in operation can be most misleading.

Peat is almost unique in possessing a solids structure which is vastly more compressible than other cohesive materials. Studies involving rates of deformation of peat must therefore involve consideration of the quasi-viscous behavior of the solids as well as the hydrodynamic phenomena of the interstitial water. Thus an investigation of the effect of load on peat cannot be reported in the form of a straightforward load-voids ratio relationship but must take the time into account as shown on attached figure. Again this relationship is subject to further correction to take into account variations of thickness of specimen.

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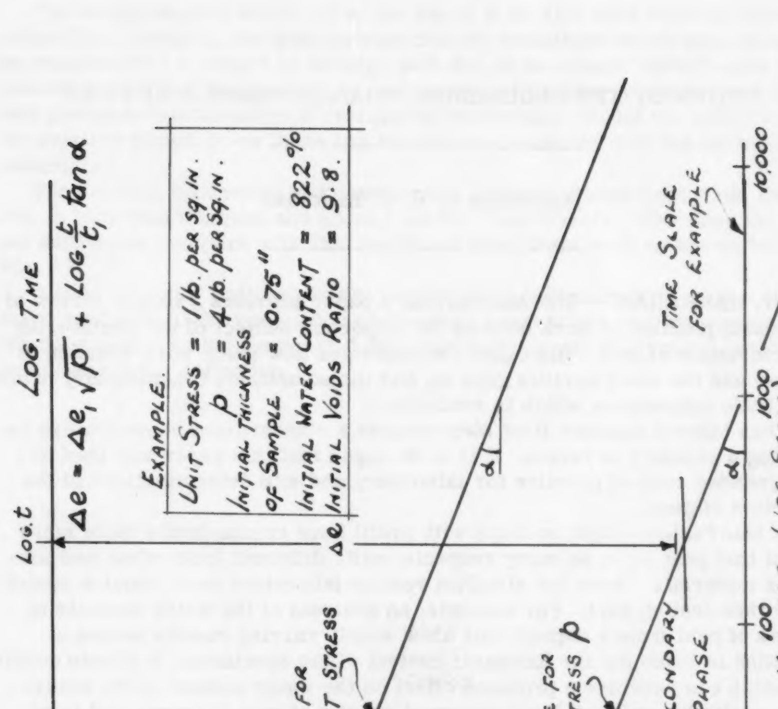
a. Proc. Paper 1937, February, 1959, by Ivan C. MacFarlane.

1. Civ. Eng. Lab. (Soil Mechanics), Univ. College, Upr. Merrion Street, Dublin, Ireland.



CHANGE IN VOIDS RATIO WITH TIME DUE TO UNIT STRESS AND STRESS p.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758 (ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767 (WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775(IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784(IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806(HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815(ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824(SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c].

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850 (SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858(SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

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JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926 (PL1), 1927(HW), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO), 1957(CO1), 1958(CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986 (IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 (SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 (HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c]. 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056 (HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065 (ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(IR2)[c], 2074(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2)[c].

JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087 (SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096 (EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134 (SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c], 2140(SM4)[c].

c. Discussion of several papers, grouped by divisions.

SM 4

AUGUST 1959 — 30
VOLUME 85

NO. SM 4
PART 2

Your attention is invited

NEWS OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS
FOUNDED 1852

JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

August, 1959

Fifth International Conference on Soil Mechanics and Foundation Engineering

The French Organizing Committee for the Fifth International Conference on Soil Mechanics and Foundation Engineering to be held in Paris July 1961 has set up the following subject grouping for papers:

Division 1 - Soil Properties and Their Measurement

a. Fundamental and Natural Properties
b. Mechanical Properties

Division 2 - Techniques of Field Measurement and Sampling
Division 3 - Foundations of Structures

a. General Subjects and Foundations other than Piles Foundations
b. Piling and Piled Foundations

Division 4 - Roads, Runways and Rail-tracks
Division 5 - Earth Pressure on Structures and Tunnels
Division 6 - Earth Dams, Slopes and Open Excavations
Division 7 - Problems not Dealt with in Divisions 1 - 6

In order that papers can be reviewed by the U. S. National Committee and sent to the French Committee for publication and distribution before the Conference, papers will have to be prepared in the near future. Each person wishing to submit a paper for publication in the proceedings of the Conference should send a statement of the contents of his proposed paper to the Secretary of the U. S. National Committee, 375 Park Avenue, Room 900, on or before October 1, 1959. Also, he should indicate the division in which the paper belongs.

As yet, the deadline for receipt of manuscripts by the French Organizing Committee has not been set. However, it appears likely that manuscripts will have to be in the hands of the U. S. Secretary by about January 1, 1960.

Note: No. 1959-30 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, SM 4, August, 1959.

U.S.S.R.—U.S.A. Soils and Foundations Engineering Seminar

The Highway Research Board of the National Academy of Sciences and the American Society of Civil Engineers, in cooperation with the Academy of Construction and Architecture, U.S.S.R. conducted four seminars in the United States during the month of June. Those meetings were held at Princeton, Massachusetts Institute of Technology, University of Illinois and the University of California, at Berkeley. The purpose of the seminars were to exchange soil mechanics and foundations information between the two countries via authorities in various fields. Professors of soil mechanics and prominent soils engineers in the region of each school were invited to the meetings. The following is a list of names of the Soviet delegation, together with their specialties:

1. I. M. Litvinov, active member of the Academy of Construction and Architecture of the Ukrainian SSR—chairman of the delegation, specialist in the field of experimental test sites and the strengthening of bases.
2. N. A. Tsytovich, active member of the Academy of Construction and Architecture of the U.S.S.R., Professor and Dr. of Technical Sciences—chairman of the National Association of Soil Mechanics and Foundation Engineering of the USSR, specialist in the field of genreal soil mechanics and of the soil mechanics of permafrost.
3. R. A. Tokar, corresponding member of the Academy of Construction and Architecture of the USSR, director of the Institute of Foundations of the Academy of Construction and Architecture—specialist in the field of bases and foundations.
4. M. M. Levkin, Chief Engineer, Department of Large Bridges.
5. A. V. Gladyrevskiy, chief specialist of Gosstroy (State Construction Administration of the USSR) specialist in the field of construction layout.

The following program of the Princeton meeting provides an indication of the type of subjects discussed, and of the format of the meetings:

Wednesday, June 3, 1959, First Session, Woodrow Wilson Hall

9:45 A.M. Introductory Remarks
Admiral W. Mack Angas, Session Chairman
Chairman, Department of Civil Engineering, Princeton University

Greetings
Mr. Fred Burggraf
Director, Highway Research Board
Mr. W. H. Wisely
Executive Secretary, American Society of Civil Engineers

Response
Academician I. M. Litvinov
Chairman, Soviet Delegation

10:10 A.M. Problems of Frozen Soil Mechanics on Engineering Practice
Academician Professor N. A. Tsytovich
U.S.S.R.

11:10 A.M. Panel Discussion
George W. McAlpin, Moderator
Chairman, Committee on Frost, Highway Research Board

Professor A. R. Jumikis
Rutgers, The State University of New Jersey

Mr. Robert Philippe
U. S. Army Corps of Engineers

Professor F. J. Sanger
U. S. Army Corps of Engineers

Professor Hans F. Winterkorn
Princeton University

12:15 P.M. Recess for Luncheon

Princeton Inn

12:30 P.M. Luncheon, Conference Room B

Professor G. P. Tschebotarioff, Luncheon Chairman

Engineering Societies in the United States
Mr. W. H. Wisely
Executive Secretary, American Society of Civil Engineers

1:55 P.M. Adjourn

Second Session

Professor Norman J. Sollenberger, Session Chairman
Princeton University

Garden Theater

2:15 P.M. Film: Vibratory Driving of Piles and Sheet Piles
Courtesy of the Soviet Delegation

2:40 P.M. Return to Woodrow Wilson Hall

2:55 P.M. Experiments with Vibratory Driving of Piles for Bridge Construction
M. M. Levkin
Chief Engineer, Department of Large Bridges, U.S.S.R.

3:20 P.M. Design of Machine Foundations
R. A. Tokar
Director, Institute of Foundations, Moscow, U.S.S.R.

3:45 P.M. Panel Discussion
Professor J. J. Slade, Jr., Moderator
Director, Bureau of Engineering Research, Rutgers, The State University of New Jersey

Professor R. K. Bernhard
Rutgers, The State University of New Jersey

Mr. Stanley J. Johnson
Moran, Procter, Mueser and Rutledge

Mr. W. P. Kinneman
Raymond International, Inc.

Professor Werner E. Schmid
Princeton University

4:30 P.M. Adjourn

About one hundred men from the Eastern area attended this meeting. Professor Tschebotarioff acted as an interpreter. The exchange of ideas was fruitful and stimulating and an air of good fellowship existed.

The Eighth Conference

Novi Sad

The eighth Conference of the Yugoslav Society Soil Mechanics and Foundation Engineering held at Novi Sad from June 27—was engaged in hearing and discussing papers submitted by members of the Yugoslav Society dealing with problems of common interest in the field of soil mechanics and foundation engineering.

Besides the three-day period set apart for the proceedings of the Conference—from June 22nd to June 24th inclusive the program provided for two days (June 25 and June 26) being spent in recreation or visiting places of interest in and around Novi Sad.

Copies of the reports are available in Serbo-Croat, with summaries in English and French. Should you desire any further information, please apply to the Secretary (Dr. ing. Dusan Krsmanovic, Trg Oslobođenja, Tehnicki takultet, Sarajevo—Yugoslavia).

Harvard Soil Mechanics Publication

Due to numerous requests for copies of the following two papers, reprints of them will be available:

Soil Mechanics Series No. 7 - "An Experimental Investigation of Protective Filters", by Bertram

Soil Mechanics Series No. 46 - "Stability Analysis of Slopes with Dimensionless Parameters", by Janbu

Copies of the publications may be obtained from:

The Gordon McKay Library
Pierce Hall
Harvard University
Cambridge 38, Massachusetts

The cost is 60¢ for No. 7 and $3.00 for No. 46.

Early Transactions Volumes Obtainable

The feasibility of reproducing the first ten volumes of ASCE Transactions (1872-1881) has been studied. It has been decided that these historic volumes could be reproduced at a cost that would permit a top price of $150 for the ten-volume set. If more than 100 engineers, or libraries, indicate an interest in obtaining such a set, the project will be undertaken. If the endeavor is successful, other rare volumes of Transactions will be reprinted.

Engineers interested in obtaining the ten-volume set should write to the Executive Secretary of ASCE, 33 West 39th Street, New York 18, N. Y.

October Newsletter

Deadline date for arrival at this office of contributions for the October Newsletter: August 20, please.

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